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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1077

A NEW SUSPENSION FOR THE CONTACT WIRES OF ELECTRIC RAILWAYS USING SLIDING BOWS.*

By Joseph Mayer, M. Am. Soc. C. E.

With Discussion by Messrs. R. D. Coombs, Charles Rufus Harte, and Joseph Mayer.

For high speeds and the high voltages generally used with them, the ordinary trolley wire suspension has proved unsatisfactory. Single or double catenary suspensions, therefore, have largely taken its place. They permit the use of longer spans, and thereby diminish the number of poles, brackets, and insulators.

With single catenary suspensions, spans of 120 to 150 ft. are mostly used. In these the contact wire is suspended at short intervals from a galvanized steel strand above, and is approximately straight and of the same length at all temperatures. Cold weather, therefore, greatly increases its tension. The amount of increase depends on the modulus of elasticity and the coefficient of expansion of the wire.

The modulus of elasticity of copper wire varies considerably. For very hard wires, it is given as high as 20 000 000 lb. per sq. in.; for moderately hard wire, such as is commonly used for contact lines, Mr. Blackwell gives 16 000 000 lb. as a fair average. The coefficient of expansion for changes of temperature is $\frac{1}{104\,400}$ per degree, Fahren-

* Presented at the meeting of February 5th 1908.

heit. The change in tension per square inch, per degree Fahrenheit, in a wire of constant length, therefore, is 191.5 lb. in the former, and 153.2 lb. in the latter case.

For 140° variation of temperature, the change in tension per square inch is 26 810 lb. in the former, and 21 450 lb. in the latter case. To avoid this variation in the tension of the contact wire, an automatic adjustment of its length has been adopted on the line, Blankenese-Ohlsdorf, near Hamburg, Germany, recently opened for traffic.

If the contact wire is firmly held by rigid suspenders without hinges, the sliding bow which rises between succeeding suspenders bends the wire in a rather sharp curve as it approaches them. At the suspenders this curve is convex downward. To prevent the jumping of the sliding bow at these curves, it must be very light, and the wire must be under high tension. At a speed of 75 miles per hour the sliding bow, with suspenders 10 ft. apart, must make eleven complete up-and-down oscillations per second. With such rapid oscillations, even if they are of very small amplitude, it is very difficult to prevent jumping and sparking; it is impossible, without springs between the bow and its heavy supporting frame, unless the bow itself is a spring.

The bending strains in the wire at the suspenders are also objectionable. To avoid large deflection of the contact wire by the pressure of the sliding bow, its minimum tension in summer must be considerable. The tension in winter is then very large. To this must be added the bending strains arising from the deformations caused by wind pressure, changes of temperature, and the pressure of the sliding bow. At high speed the latter is very variable because the bow oscillates rapidly. The consequent bending strains in the contact wire are greatly reduced, in the Blankenese-Ohlsdorf line, by the use of a supplementary steel wire some distance above and parallel with the copper contact wire. The contact wire is hung by loops, 3 m. apart, from this steel wire, and can rise at these loops, thus permitting the wave raised by the sliding bow to pass unhindered.

The supplementary wire, which, like the contact wire, is nearly horizontal at all temperatures, is carried by suspenders, at intervals of 6 m., from a steel strand above having considerable deflection. The contact wire is grooved, and 100 sq. mm. in cross-section, the steel wire is 6 mm. in diameter, and the steel strand above is of seven

wires, having a total cross-section of 350 mm. With a small contact pressure, the lifting of the wire by cross-winds and the passing sliding bow may not be sufficient to permit the latter to touch the supplementary wire and the suspenders. The whole design is ingenious, but complicated and expensive to install, and probably troublesome to maintain. It shows a clear appreciation, resulting from past experience, of the main defects of the catenary suspensions, namely, the excessive tension in the contact wire in winter, the large bending strains in the wire at the suspenders produced by the sliding bow, and the jumping of the latter.

The details of the automatic adjustment of the tension of the contact wire by counterweights are complicated, due to the necessity of insulation, and the impossibility of running the wire over pulleys of moderate size without excessive bending strains. The writer is not aware that such details have been published. The spans of the carrying strand are 48 m., and steady braces are used at the brackets to prevent any lateral deflection of the contact wire there. Large bending strains in the contact wire, at these points and probably at the automatic adjustment of its length, due to lateral deflection by wind pressure, are inevitable. That such complicated and expensive contrivances have been adopted, in the country having the longest experience with catenary suspensions, proves that the much simpler structures previously in use have not been satisfactory even with the moderate speeds used thus far. This is also confirmed by the recent adoption of very short spans without ropes, but with two wires per phase, tied together at short intervals, for the approaches of the Simplon Tunnel.

Since the adoption of steel strands for carrying the contact wire leads to such complicated contrivances, it is worth while to search for a simpler solution of the problem. A clear statement of it will be helpful for the purpose.

The contact wire serves as the track for a rapidly moving sliding bow. It cannot be straight because it must pass at an elevation of about 16 ft. under overhead crossings and at an elevation of about 24 ft. over grade crossings. The horizontal curvature gives no trouble in this respect, as long as the wire does not pass beyond the range of a sliding bow of moderate length. The sliding bow, however, must follow the vertical curvature of the wire. This results in centrifugal

force and consequent variation of the contact pressure. The amount of centrifugal force is proportional to the square of the train speed and to the weights of the parts of the sliding bow and its supporting frame moving in curves, and inversely proportional to the radii of curvature of their motion. Where the curvature of the motion of the sliding bow is convex upward, the contact pressure is increased; where it is convex downward, it is reduced. The increase, if acting on the wire, should not be large enough to produce dangerous bending strains, and the reduction should not produce interruption of contact and consequent sparking.

The contact wire is exposed to tensions and bending strains, produced by weights, wind pressures, pressure of the sliding bow, and changes of temperature. The sum of these should never, and nowhere, exceed a safe limit. These ends should be attained by the simplest possible means. Simplicity demands the use of long spans, and these require large deflections in the contact wire for preventing excessive tensions. With large vertical deflections, strong winds produce large horizontal deflections. The length of the practicable sliding bow fixes the permissible horizontal deflection. Large and round wires are deflected less by wind than small and grooved wires. The permissible vertical deflection, therefore, is influenced by the practicable length of the sliding bow and the shape and size of the wire. Drop of temperature shortens the wire, reduces its deflection, and increases its tension; it thereby reduces the safe span.

An adjustment which will lengthen the wire in winter will evidently permit the safe use of longer spans. Large deflections result in steep terminal slopes of the contact wire, and a large variation of this slope is produced by changes of temperature and by the lifting of the wire by the passing sliding bows. The large lateral deflections produce large lateral slopes of the wire at the clamps. If the wire leaves the clamps in a fixed direction it will be bent up and down by changes of temperature and the passing sliding bows, and to the right and left by cross-winds. This bending of the wire at the ends of the clamps results in bending strains which are large in long spans and are the principal reason why such spans are impracticable with ordinary clamps.

If long spans are to be used with safety, these strains must be reduced. Due to the variable vertical terminal slope of the wire, the

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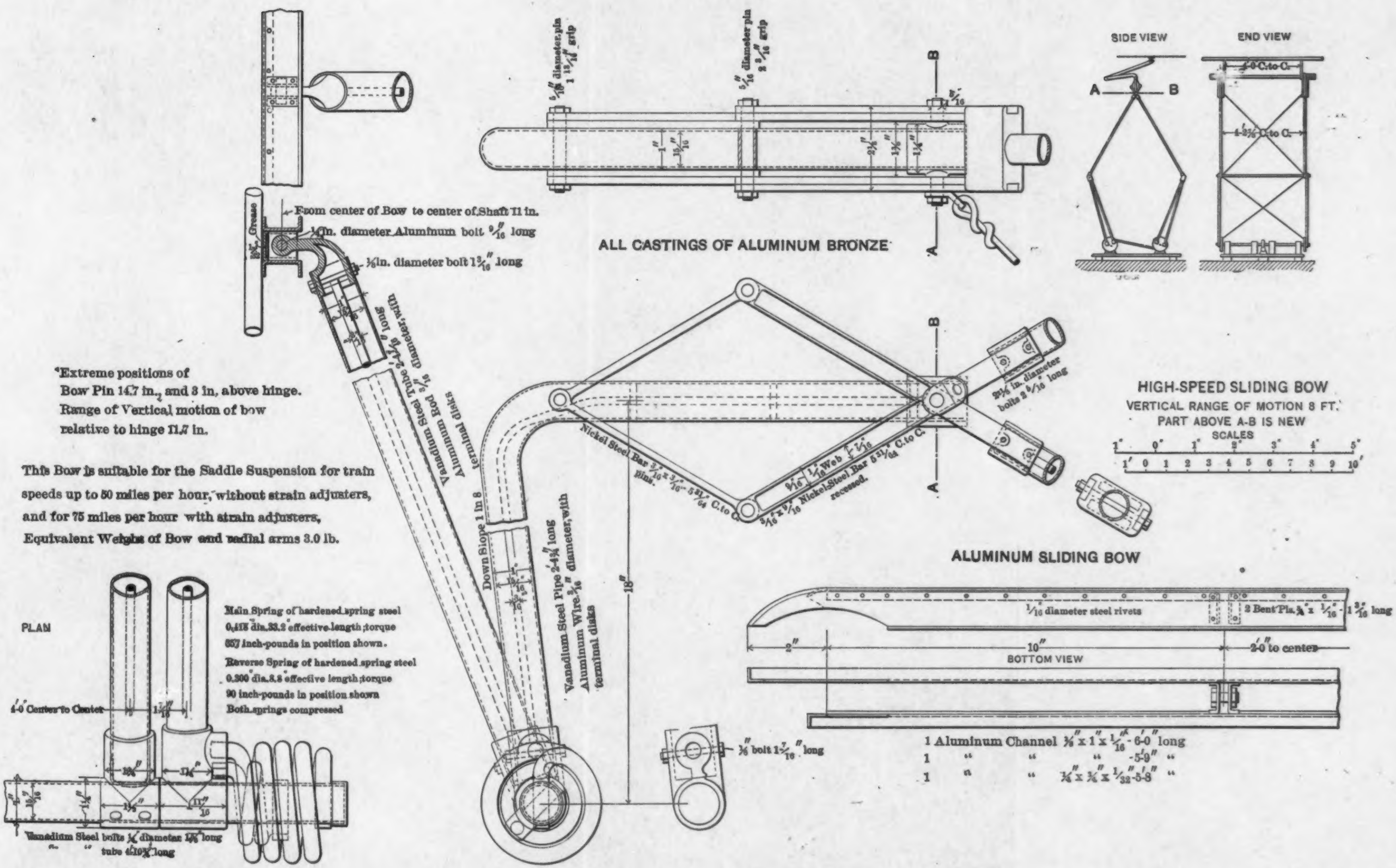
SUSPENSION FOR WIRES OF ELECTRIC RAILWAYS.



sliding bow approaches the clamp with a variable upward slope of its motion. It should pass the clamp along a curve of large radius tangent to this variable slope of approach. This is not practicable if the wire leaves the clamp in a fixed direction.

The reduction of the bending strains in the contact wire at the clamps and the creation of a smooth track, of large curvature, at all temperatures, for the sliding bow, is obtained by the suspender shown on Plate I. It consists of four castings firmly bolted together and provided with a central ear for attaching it to an insulator pin. The wire is firmly held in the central part of the suspender. It leaves this central part 8 in. long with vertical slopes of 3% below the horizontal. The suspender is designed for 240-ft. spans of 000 B. & S. gauge wire, with 2½ ft. maximum vertical deflection. It is intended for use on a line having maximum train speeds not exceeding 80 miles per hour.

The variation of temperature is assumed at 140° fahr. Strain adjusters, one mile apart (to be described later) in effect reduce this variation to 84° fahr. The central part of the suspender encloses the wire. The sliding bow moves here, along the bottom of the suspender, along an arc of a circle having a radius of 11 ft. 1⅓ in. Where the wire leaves the central part, its bottom is tangent to the bottom of the suspender. Next to the central part of the suspender are two channels, open below. These channels have top walls which, in effect, are horizontal cylinders of 75 ft. radius. The top walls are tangent to the top of the wire where it leaves the central part. The walls are recessed, for easier manufacture. The side walls, which are only 14 in. long, are vertical cylinders of 70 ft. radius. They are tangent to the sides of the wire where it leaves the central part. Below these terminal channels of the suspender the sliding bow moves along the bottom of the wire. The lower edge of the side walls, for all positions of the wire, is above its bottom, so that the side walls will not interfere with the sliding bow. On curves, the suspender is hung with its bottom parallel to the plane of the track.

The wire is assumed to have an ultimate strength of 50 000 to 60 000 lb. per sq. in. and an elastic limit of 40 000 to 45 000 lb. The modulus of elasticity is assumed at 16 000 000 lb. With these assumptions, the largest tension in the wire is 20 850 lb. per sq. in. The largest bending strain at the same time is 5 340 lb. This gives a total

maximum strain of 26 190 lb. No ice was assumed on the contact wire. Ice averaging ¼ in. in thickness, increasing the diameter by ½ in., would increase the tension to 22 450 lb. per sq. in.

These strains give to the contact wire about the degree of safety of first-class railroad bridges. They amount to about as much as the maximum tensions, exclusive of bending strains, in the contact wires of the best catenary suspensions of the United States.

For a speed of 80 miles per hour, the equivalent weight of the sliding bow should not exceed 2½ lb., that is, the sliding bow and its frame should produce no larger centrifugal force than a weight of 2½ lb. moving like the contact point. With a wind pressure of 12 lb. per sq. ft. of wire, counting diameter into length as the area, its largest lateral deflection is 1.8 ft.

To avoid cutting a groove in the sliding bow, the suspenders must be placed alternately to the right and left of the center line of the track. The sliding bow must have an effective length of 4 ft.

The suspender is similar in its action to the saddle carrying a cable of a suspension bridge. It is an inverted saddle, open below, and provided with sides and a roof against which the wire rests when it is deflected upward or to the sides. The term, saddle suspension, therefore, appears to be an appropriate name.

The strain adjuster, shown on Plate II, consists essentially of two half suspenders, each cast in two pieces, and carrying cross-heads which are guided by a stout steel bar and moved by steel or bronze screws. The half suspenders change the direction of motion of the sliding bow to a horizontal one. Two properly shaped plates form the track for the sliding bow between the two half suspenders. The whole is attached to a suitable frame, and is carried by insulators from a bridge crossing the tracks. The distance between the two half suspenders can be varied about 2 ft. 8 in. by turning the screws. The brackets carrying the other suspenders must be hinged at the posts, so as to permit free motion of the suspenders within a limited range, approximately in the direction of the track, without changing the direction of the suspenders. The tension in the contact wire will then automatically readjust the position of the suspenders and the lengths of the spans when the screws of the strain adjusters are turned. The same end can also be easily attained by span wire construction.

PLATE II.
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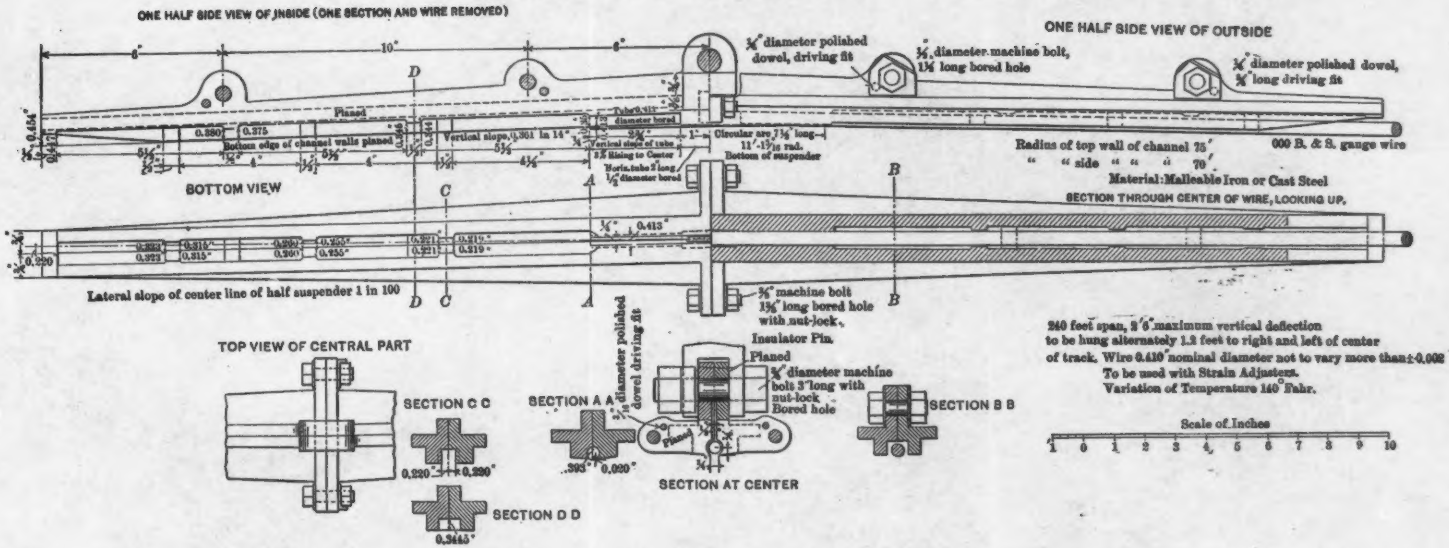
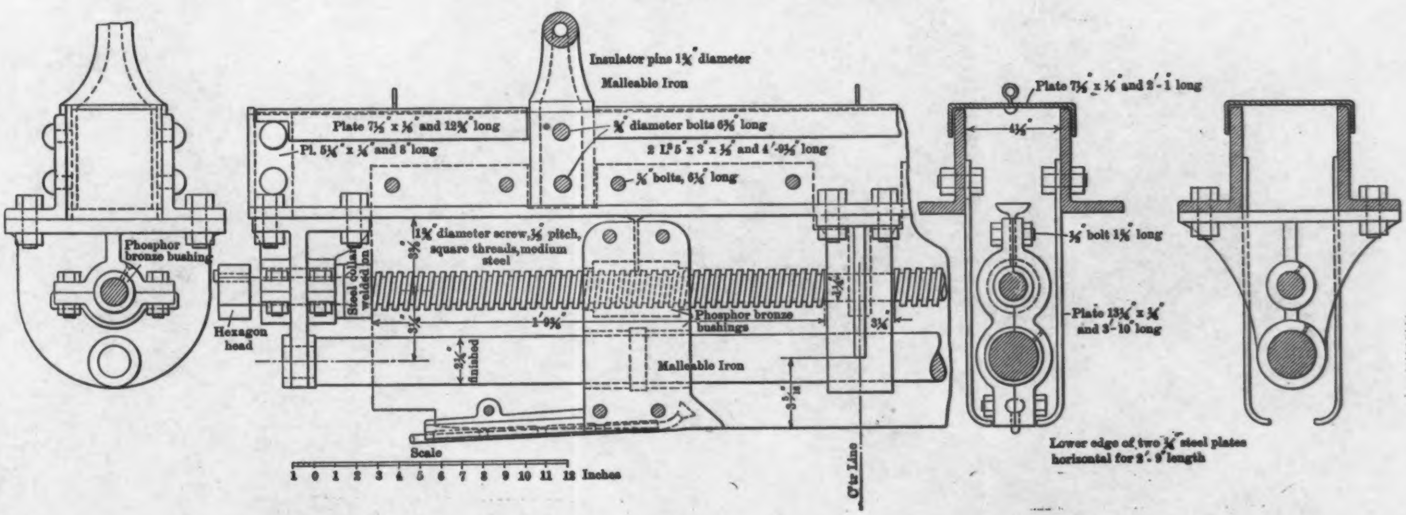
SUSPENDER FOR 000 B. & S. GAUGE WIRE



STRAIN ADJUSTER FOR CONTACT CONDUCTOR



The bridge carrying the strain adjusters and its supporting posts should be of sufficient strength to serve as an anchorage in case the contact wire of an adjoining span is broken by a derailment overturning a post. The swinging brackets or span wires, in this case, will probably prevent the overturning of other posts. The adjustment must take place once or twice in the fall, when the cross-heads are let out, and as often in the spring, when they are pulled together. It will generally be possible to reduce the variation of temperature with one adjustment to 84° fahr. or less. These strain adjusters effect a large reduction of the maximum tension in the contact wire, and thereby make possible the safe use of longer spans with moderate maximum deflections. An automatic adjustment producing constant tension in the contact wire would be desirable, and is attempted in the Blankenese-Ohlsdorf line. It is reported that the contact wire is run over pulleys to the side of the track and attached to counterweights. The wires used are probably grooved, and at least 0.2 in. wide. Running such a wire over a pulley 25 ft. in diameter results in a bending strain of 10 667 lb. per sq. in. This balances the reduction in tension obtained. To gain any advantage, a much larger pulley must be used.

The writer believes that large bending strains in the contact wire cannot be avoided in any adjustment appliance, without using substantially the suspender here described. After a number of futile attempts to find something more perfect and automatic than his strain adjuster, he arrived at the conclusion that a further reduction of the maximum tension in the contact wire cannot be obtained at moderate cost by any sufficiently simple and reliable strain adjuster.

With the catenary constructions, large deflection of the contact wire by the sliding bow must be prevented by the stiffness and tension of the wire. To obtain these safely, a large wire is essential for high speed. With the saddle suspension, 0 B. & S. gauge wire is fully adequate to obtain the requisite mechanical strength; therefore, with it, the size of the wire is mainly governed by the needed electrical conductivity.

On a superficial consideration it would appear that it is more difficult to make a sliding bow that will rise and fall 1 or 2 ft. during the passage of a 240-ft. span than to make one that will rise and fall ½ in. or 1 in. in a span of 10 or 13 ft. The opposite, however, is true.

The sliding bows used with catenary suspensions must be able to follow the wire at overhead and grade crossings; therefore, they must have a range of vertical motion of about 8 ft. Unless the approach slopes to these crossings are made very long, the sliding bows must be able to follow the contact wire around a considerable vertical angle.

With the saddle suspension, the sliding bow must follow a vertical angle in the contact line at every suspender. The necessary ability to do so is used more often with the saddle suspension. The sliding bow described in Appendix C can be used for the highest railway speeds with a properly designed saddle suspension.

The sliding bow for the catenary suspension must also be fit for extremely rapid oscillations of small amplitude. In oscillations of the same amplitude, the maximum velocity of the oscillating body is proportional to the number of oscillations per second, and the maximum acceleration is proportional to the square of this number. The maximum velocity and acceleration are both proportional to the amplitude. The variation of the contact pressure is proportional to the maximum vertical acceleration of the sliding bow. With the same law of oscillation, the same maximum acceleration would be obtained if the amplitude were inversely proportional to the square of the number of oscillations per second. In this case the amplitude of the large and slow oscillations with the saddle suspension and that of the small and rapid oscillations with the catenary suspensions are about inversely proportional to their numbers per second. The laws of the two oscillations are not the same, but it is evident that, with the suspenders here described, the small and rapid oscillations of the catenary suspensions require a larger upward acceleration of the sliding bow, and therefore produce a larger reduction of the contact pressure.

With the catenary suspensions, the calculation of the variation of the contact pressure is extremely complicated, and, the writer believes, has never been accurately performed. With a properly designed saddle suspension, the calculation is simple, the shortest radii of the curvature of the motion of the sliding bow being given in the design of the suspender.

The foregoing gives the results of the writer's investigations, the proofs are given in the appendices.

Reviewing the problem: The aim is a safe contact wire, cheap to construct and maintain, offering a suitable track for a rapidly moving

sliding bow. The wire is safe if exposed to moderate maximum strains not exceeding three-fourths of its elastic limit. To prove its safety, the bending strains and tensions produced by the incident forces and the changes of temperature must be calculated.

The suspender here described is imperfect in several respects:

First. It is unsuitable for lines using trolley wheels.

Second. Since the wire in the terminal channels is convex downward, the sliding bow, for high speed, to avoid jumping, must be light, and the static contact pressure considerable. For the highest speeds, the use of strain adjusters is essential.

Third. The wire at the center of the suspender must be sharply bent so that it receives a permanent kink. This is inconvenient if, for any reason, a readjustment is desired.

Fourth. The remaining bending strains in the wire are about 8 000 lb. per sq. in. when strain adjusters are used, and more than 10 000 lb. per sq. in. without strain adjusters. This, with the tension, gives a total strain of from 26 000 to 28 000 lb. per sq. in. of wire. This is considered excessive by some engineers.

Fifth. Many different shapes of suspender are required to meet the varying conditions, and great accuracy of manufacture is needed to obtain the proper curvature of the wire.

For these reasons, the suspenders are very expensive. A simpler and much cheaper suspender, avoiding all these defects and giving bending strains in the wire only about half as large, will be described later.

Appendix A gives the derivation of approximate formulas for calculating the bending strains produced by normal forces in a wire under tension.

Appendix B calculates the maximum coexisting bending strains and tensions for a particular case of the saddle suspension; it also gives the bending strains and tensions for a suspension with ordinary clamps.

Appendix C gives the theory and description of a sliding bow, suitable, with the highest train speeds, for the saddle suspension. In previous pamphlets, entitled: "Overhead Contact Lines" and "The Saddle Suspension," the writer has described the same problem, and has there compared the different suspensions. His views on several details have since been modified by a study of the action of sliding bows.

APPENDIX A.

In many designs of suspensions, the bending strains in the contact wire far exceed the tensions. Their neglect makes many inferences from calculations of the tension only utterly misleading.

The writer has developed formulas for calculating the bending strains in the contact wire produced by a known pressure of the sliding bow, and those at the ends of the clamps firmly holding the wire in a fixed direction produced by lateral and vertical deflections. For round copper wires having a modulus of elasticity of 16 000 000 lb. per sq. in. the formulas are as follows:

For 0000 B. & S. gauge wire $S = 10\,900 \dfrac{Q}{\sqrt{T}}$

" 000 " " " " $S = 11\,800 \dfrac{Q}{\sqrt{T}}$

" 00 " " " " $S = 12\,700 \dfrac{Q}{\sqrt{T}}$

" 0 " " " " $S = 13\,600 \dfrac{Q}{\sqrt{T}}$

Q is the pressure of the sliding bow, in pounds;
T is the tension in the wire, in pounds;
S is the bending strain per square inch.

The formulas are approximately correct for the usual values of Q and T.

In a wire held in a clamp and exposed to a tension, T, differing in direction from that of the wire where it issues from the clamp there arises a bending strain largest at the end of the clamp. If the tension, T, is decomposed into a component acting in the direction of the issuing wire and a component, N, normal to it, then the above formulas give the bending strain per square inch for $Q = 2\ N$.

The proof for these formulas for wires with a modulus of 20 000 000 lb. per sq. in. is given in the writer's pamphlet, "Overhead Contact Lines." S is approximately proportional to the square root of the modulus of elasticity.

APPENDIX B.

Calculation of a suspender for a 240-ft. span of 000 wire with 2 ft. 6 in. maximum vertical deflection, for a train speed of 80 miles per hour. Variation of temperature 140° fahr., reduced in effect to 84° fahr. by strain adjusters.

By numerous trial calculations, it was found that, with the assumed speed, the design of a suitable sliding bow is difficult with larger deflections. Longer spans give maximum strains in the con-

tact wire which, though much smaller than those in the contact wires of most catenary suspensions, are believed to be inadvisable at the start. A large factor of safety is possible with this suspension without excessive cost and complication; therefore, it has been the governing consideration in selecting the design.

With a deflection of 2 ft. 6 in., the vertical terminal slope of the wire is 4.17 per cent. A train running at 80 miles per hour will have generally two sliding bows; these lift the wire and approach the suspender at a much flatter slope. The weight carried by the wire to the suspender is reduced by the two approaching bows by an amount depending on their contact pressure and distance apart. With a contact pressure of 15 lb., which is needed for preventing jumping at the suspenders, and 30 ft. distance of the two sliding bows, the weight carried by the wire to the suspender is decreased, at the arrival of the first bow, from 60.8 lb., the weight of a half span of wire, to 32.7 lb. The slope of approach of the sliding bow is reduced in the same ratio, or to $4.17 \times \frac{32.7}{60.8} = 2.24$ per cent. The vertical component of the velocity of the sliding bow at 80 miles per hour is then $117.33 \times 0.0224 = 2.63$ ft. If there is occasionally only one sliding bow, this upward velocity of approach to the suspender is:

$$4.17\% \times \frac{45.8}{60.8} \times 117.33 = 3.14\% \times 117.33 = 3.678 \text{ ft.}$$

As shown in Appendix C, this can be easily taken care of by the sliding bow there described.

The length of one span of the contact wire is given by the formula:

$$S = L + \frac{8}{3}\frac{D^2}{L} = 240 + \frac{8 \times 2.50^2}{3 \times 240} = 240.0694 \text{ ft.}$$

The tension due to the weight of the wire is:

$$T = \frac{0.507 \times 240^2}{8 \times 2.5} = 1\,460.2 \text{ lb.}$$

The stretch produced by this tension is:

$$\frac{1\,460.2 \times 240}{0.1317 \times 16\,000\,000} = 0.1663 \text{ ft.}$$

The length of the wire without tension, therefore, is 239.9031 ft. The tension in the wire at maximum temperature with a wind pressure of 12 lb. per sq. ft. is:

$$\frac{0.652 \times 240^2}{8\ \delta} = \frac{4\,694.4}{\delta},$$

where δ is the deflection. The stretch by this tension is $\frac{0.5347}{\delta}$ ft. Equating the length with the deflection, δ, to the length without tension plus the stretch, we obtain:

$$\delta^3 + 8.721\ \delta = 48.12.$$

And from this, $\delta = 2.854$ ft.: This gives a tension, with maximum temperature and wind pressure, of 1 645 lb., or 12 490 lb. per sq. in. The greatest lateral deflection is:

$$2.854 \times \frac{0.410}{0.652} = 1.8 \text{ ft.}$$

By a similar equation, the deflection at minimum temperature with a wind pressure of 6 lb. per sq. ft. is found to be 1.434 ft., and the tension 2 746.5 lb., or 20 850 lb. per sq. in. With ice on the wire, ¼ in. thick, and a wind pressure of 6 lb. per sq. ft. on the enlarged wire, the deflection is 2.06 ft., and the tension 2 956 lb., or 22 450 lb. per sq. in. The least vertical deflection occurs at minimum temperature with wind and without ice; it is 1.33 ft. This gives, for the least vertical terminal slope of the wire, 2.66 : 120 = 2.22 per cent.

The maximum vertical terminal slope was found to be 4.17 per cent. To allow for the possible inaccuracy in the erection, and in the modulus of elasticity of the wire, and for small changes of grade, the writer will assume a variation of $\pm 0.24\%$ from the calculated minimum and maximum terminal slopes. This makes the new maximum slope 4.41 per cent. If the tube is made to slope 3%, then the wire hangs down 1.41% below it. This results in a bending strain. The tension with maximum vertical slope is 1 460 lb. The component normal to the tube is:

$$0.0141 \times 1\,460 = 20.65 \text{ lb.}$$

In Appendix A it is shown that the corresponding bending strain in the wire is:

$$\frac{23\,600 \times 20.65}{\sqrt{1\,460}} = 12\,730 \text{ lb. per sq. in.}$$

This occurs together with a tension of 11 088 lb., giving a total strain of 23 818 lb. per sq. in.

As the tension increases, by the reduction of the vertical deflection, this bending strain rapidly decreases. The largest lateral deflection was found to be 1.8 ft.; this corresponds to a terminal lateral slope of 3 per cent. If the radius of the side walls of the terminal channels is made 70 ft. and their length 14 in., then the terminal tangent to the side wall has a lateral slope of 14 : 840 = 1.67 per cent. If the wire rests against the side wall, it leaves the suspender with this lateral slope. The difference between this and 3% is 1.33%, and produces a bending strain in the wire. The tension at the same time is 1 645 lb., or 12 490 lb. per sq. in. The normal force acting on the wire is 1 645 × 0.0133 = 22 lb. The corresponding bending strain is:

$$\frac{22 \times 23\,600}{\sqrt{1\,645}} = 12\,800 \text{ lb.}$$

This would produce a lateral curvature with a radius of 21 ft. 4 in.

The sliding bow, at the same time, may produce a bending strain of:

$$\frac{15 \times 11\,800}{\sqrt{1\,645}} = 4\,365 \text{ lb.}$$

The resultant of the two bending strains is:

$$\sqrt{12\,800^2 + 4\,365^2} = 13\,530 \text{ lb. per sq. in.}$$

This, together with the tension, gives a total of 26 330 lb. The actual bending strain is much less than the amount here calculated, because the wire, being bent at the end of the side wall to a shorter radius than that of the side wall, lifts off it and issues from the suspender with a larger lateral slope than assumed. This greatly reduces the lateral bending strain and leaves a safe margin for error in erection.

The least vertical end slope of the contact wire when not affected by sliding bows was found to be 2.22%, or, allowing for 0.24% error from various sources, 1.98 per cent. This is reduced by the raising of the wire by two approaching sliding bows. A sliding bow with a contact pressure, Q, at the distance, A, from the end of a span, L, diminishes the weight carried by the wire to that end by $\frac{L-A}{L}\,Q$. If Q is 15 lb., and there are two sliding bows 30 ft. apart, one at the suspender, then the weight carried by the wire to the suspender is decreased by 28.12 lb. The tension in the wire with minimum slope is 2 746 lb. The terminal slope, therefore, is reduced 1.02 per cent.

This reduction is at one end of the suspender only. The suspender is hung from a horizontal bolt, and can swing freely around it. It will rise toward the approaching sliding bows, reducing the relative change of slope due to this cause to one-half, or 0.51 per cent.

The proper minimum slope for the calculation, therefore, is 1.47 per cent. The difference between this and the tube slope is 1.53 per cent. To produce this difference of slope by a curved top wall of 900 in. radius requires a length of $900 \times 0.0153 = 13.77$ in. Therefore, 14 in. is ample, with a freely swinging suspender. The 18 in. adopted is adequate for a fixed suspender.

The suspender will turn with every passing sliding bow; therefore, it is very improbable that it will get stuck by rust if it has at the start adequate play at the hinge.

The wind pressure and the lateral displacement of the suspenders, however, produce a lateral force which is a source of considerable friction at the hinge. Reasonable doubt about the freedom of motion, therefore, may be entertained, and the safest course, before experience, is to assume the suspender fixed and to make the terminal channels 18 in. long.

Provision might be made for lubricating the hinge, and then it

would be safe to reduce the length of the suspender or to increase the radius of the top wall. The latter change would permit the use of a heavier sliding bow without resultant sparking.

For determining the maximum strain in the contact wire, it is necessary to know the bending strains that occur at the same time with the maximum tension. In this case, the wire rests against the top and side walls, and there are no bending strains, outside of the suspender, except from the pressure of the sliding bow.

In the terminal channels, the bending strains are due to lateral and vertical bending with known radii. The strain due to lateral bending is:

$$\frac{0.205 \times 16\,000\,000}{840} = 3\,905 \text{ lb.}$$

That due to vertical bending is:

$$\frac{0.205 \times 16\,000\,000}{900} = 3\,644 \text{ lb.}$$

The combined bending strain is:

$$\sqrt{3\,644^2 + 3\,905^2} = 5\,340 \text{ lb.}$$

If the terminal channels were omitted, the bending strain due to wind pressure would be 30 300 lb. The vertical bending strain would be about one-half as much, and the resultant of the two, 34 000 lb. per sq. in. This, with 12 800 lb. tension, would give a maximum strain of 46 800 lb. The suspension would be unsafe, even with the 84° variation of temperature here assumed. Without strain adjusters, the tensions, or the bending strains, or both, would be vastly increased. At high speeds, the suspender would receive a severe blow and the wire a violent jerk with every passing sliding bow. Most sliding bows would be broken.

It is of interest, in obtaining a practical confirmation of this theory, to calculate the strains in the contact wire of an ordinary trolley wire suspension for a span of 100 ft. of 000 B. & S. gauge, round wire, for variations of temperature of 140° fahr., with a wind pressure of 12 lb. per sq. ft. in summer, and 6 lb. per sq. ft. in winter.

Trial calculations show that a maximum deflection of about 1½ ft. gives the smallest maximum strain in the wire. A down slope of 1½% where the wire leaves the clamp is most favorable.

The strain, with maximum temperature and wind pressure, is 4 000 lb. tension and 27 750 lb. bending, giving a total strain of 31 750 lb. per sq. in. At minimum temperature, with wind, and with two sliding bows of 15 lb. contact pressure 30 ft. apart, the tension is 15 600 lb. and the bending strain 16 900 lb., giving a total strain of 32 500 lb. per sq. in. The terminal down slope of the wire, at maximum temperature, without wind, is 6%; at high speed, this, though reduced by the approaching sliding bow, when without adequate

transition curve, is dangerous to the wire, the clamp, and the sliding bow. The suspension, therefore, is only suitable for moderate speeds. The above maximum strains are only moderately safe. These results agree with actual experience. With smaller wires, the lateral deflections and the strains are larger, and the degree of safety is reduced.

The saddle suspension for 240-ft. spans, here proposed, gives much smaller maximum strains in the contact wire, and, for all deflections, provides transition curves of large radius which make high speeds harmless.

APPENDIX C.

SLIDING BOWS FOR HIGH-SPEED ELECTRIC RAILWAYS.

The contact wire which forms the track for the sliding bow is suspended either at short or long intervals. In the former case, the weight of a half span of contact wire is generally much less than the upward pressure of the sliding bow, called the contact pressure. The wire, therefore, is bent up by the bow beyond the supports. In the latter case, the wire of a half span weighs more than the amount of the contact pressure; it is lifted by the bow from its normal sag to a position still below the horizontal. In all cases, the bow moves along vertical curves. At high speeds this produces a considerable amount of centrifugal force, which, according to its up or down direction, increases or reduces the contact pressure. The increase should not be so large as to produce dangerous bending strains in the wire; the reduction should not be so large as to cause separation of the bow from the wire and consequent sparking. The centrifugal force is proportional to the weights of the parts of the bow and of its supporting frame which move in curves, and to the square of their speed, and inversely proportional to the radii of curvature of their motion. It is given by the equation:

$$C = \Sigma \frac{w\, v^2}{g\, R} = \frac{v^2}{g} \Sigma \frac{w}{R}$$

where C is the centrifugal force, in pounds, v is the train velocity, in feet per second, g is 32.2 ft., $\frac{w}{R}$ is the weight of a part of the sliding bow or of its frame divided by the radius of its motion.

$\Sigma \frac{w}{R}$ is the sum of all these quotients for the sliding bow and its frame. If $\Sigma \frac{w}{R} = \frac{W}{r}$, where r is the radius of curvature of the motion of the contact point, then W is the equivalent weight of the sliding bow and its frame; it is the weight which, if moving as the

contact point, produces the same centrifugal force as the sliding bow and its frame. With the saddle suspension, r, at the center of the suspender, is conveniently made from 5 to 12 ft. Here the sliding bow presses against the suspender, and a large contact pressure will not endanger the wire. In the terminal channels of the suspenders the wire is generally convex downward, and the centrifugal force reduces the contact pressure. r can here be made 75 ft. with, and 40 ft. or more without, strain adjusters. For a speed of 80 miles per hour, an equivalent weight, W, of 2.5 lb. and a radius, r, of 75 ft., the centrifugal force is $C = \frac{2.5 \times 117.33^2}{32.2 \times 75} = 14.25$ lb. A minimum static contact pressure of 15 lb. will be reduced by the centrifugal force to 0.75 lb., and will be adequate to prevent interruption of contact at the suspenders.

For a speed of 50 miles per hour, an equivalent weight, W, of 3 lb., and a radius, r, of 40 ft., the centrifugal force is $C = \frac{3 \times 73.33^2}{32.2 \times 40} = 12.52$ lb. A static contact pressure of 15 lb., therefore, is ample to prevent interruption of contact at the suspenders.

Plate I shows a sliding bow of 3 lb. equivalent weight when 6 ft. long with 8 ft. range of vertical motion. The bow is made of three aluminum channels, and is 6 ft. long, over all. It has a shallow space for grease, which latter must be sticky so as not to be thrown out by centrifugal force. The bow proper is connected by hinges to two radial arms. The latter are bolted to a tubular shaft. The shaft rests in bearings carried by the ends of two bent tubes. Each of the bent tubes is held by a pin and a sleeve. The sleeve forms the upper hinge of the main diamond frame. This diamond frame has extensions upward giving support to the top of the tubes. The shaft carrying the radial arms also carries at each end two springs. The free ends of the larger or main springs are inserted into holes in the hubs of the bent tubes. The free ends of the smaller springs rest against the lower sides of these hubs. The main springs press the radial arms up; the smaller reverse springs press them down.

The frame below *A-B* can be designed so that its upward pressure, when it rises with uniform speed, is approximately constant and produces a contact pressure of 15 lb. for sliding bows intended for maximum speeds of 50 miles an hour. The contact pressure required to make the frame fall with uniform speed is 15 lb. plus twice the friction. Assuming the friction at 2 lb., the contact pressure with uniformly falling frame is 19 lb.

In the following 15 lb. is called the active or minimum static contact pressure, and 19 lb. the passive or maximum static contact pressure. The equivalent weight of an ordinary diamond-frame sliding

bow can be greatly reduced by using an aluminum bow and light vanadium steel tubing with McAdamite connection castings. It will hardly be possible, however, to make it less than 6 lb. for a bow with 8 ft. range of vertical motion. This, as above shown, is too much for the highest speeds.

With the writer's modified frame, the two radial arms carrying the bow are ordinarily in an intermediate position, produced by proper adjustment of the springs. When the bow approaches the suspender of the saddle suspension it rises, together with the frame, the contact pressure being 15 lb. After the bow passes the suspender, the frame continues to rise; the bow itself, which rose as it approached the suspender, is made to fall as it passes the central part of the latter. The contact pressure at the central part of the suspender is between the latter and the bow, and, at high speeds, exceeds 100 lb. Beginning at the suspender, the radial arms turn down, and the torque of the springs increases. This increasing torque of the springs determines the amount of the contact pressure. Due to the presence of these springs, the contact pressure depends on their rigidity and the direction of the radial arms. The heavy frame below the springs, by its inertia, continues to move up, but with a retarded motion, because the increasing contact pressure (together with the weights) is larger than the constant force pushing it up. After the upward velocity of the frame is reduced to 0 (which occurs, as will be shown, about 0.15 sec. after passing the bow), it begins to move down with an accelerated motion. The radial arms, which have been turning down, begin shortly after to turn up again, and they reach not only their original angle of slope, but rise beyond it. The contact pressure decreases to less than the minimum static pressure, and the downward motion of the frame is retarded; it soon stops, and begins to go up again. After a few oscillations, the damping action of the friction stops the swinging of the radial arms.

It is evident, from the foregoing, that the sliding bow and the radial arms alone follow closely the sharp curves in the motion of the contact point. The sliding bow follows the curves accurately, the parts of the radial arms nearer the shaft move in a different manner and their weight is less important than that of the bow.

The writer, in his paper entitled "Overhead Contact Lines," has shown that the radial arms are equivalent to a weight one-third as large moving like the sliding bow. The sliding bow proper weighs 1.72 lb. The equivalent weight of the sliding bow and radial arms is 3 lb. In this design the two radial arms are bolted to the shaft, they therefore always move through the same angle. The whole sliding bow moves like the contact point. The equivalent weight of the frame largely determines, as will appear from the following calculations, the needed range of relative vertical motion of bow and shaft.

The radial arms sometimes point in the direction of the train motion, sometimes backward.

There may be a strong head wind. The wind pressure on the sliding bow is then due to the sum of the velocities of wind and train. If the radial arms point forward, this wind pressure increases the upward slope until it is balanced by the change in the torque of the springs. The amount of the contact pressure is not directly affected thereby. If the radial arms point backward, the wind pressure reduces their upward slope until it is balanced by the changed torque of the springs. In the former case the remaining available upward range, in the latter the downward range, of motion is reduced. It is desirable, therefore, to limit as much as possible the change in the slope of the radial arms produced by wind pressure. As will appear from the following calculations, this is attained by the use of reverse springs. The action of the reverse springs is limited to only the upper part of the range of motion of the radial arms.

The reverse springs used in the design here described have the same stiffness as the main springs. The change in torque for a given arc of motion, therefore, is twice as much within the range of their action as outside of it.

For showing the range of adaptation of the sliding bow, the writer will assume an unfavorable case, namely: A span of 220 ft., a maximum vertical deflection of 4 ft., a 000 B. & S. gauge wire, and a train speed of 50, with an occasional head wind of 80, miles an hour.

The radial arms are 30 in. long from center of bow hinge to center of shaft. The springs are adjusted so that the bow hinges, which are on the center line of the sliding bow, are 11 in. above the center line of the shaft when the contact pressure is 15 lb., and the train is moving at a speed of 50 miles an hour, while there is no wind, and the radial arms are pointing backward. In this case the wind pressure on the bow and the radial arms is due to the train velocity only; it is equivalent to 3.38 lb. acting on the bow. The weight of bow and radial arms is equivalent to 2.75 lb. at the bow and 1.85 lb. at the shaft. The horizontal distance between the bow hinges and the shaft is:

$$\sqrt{30^2 - 11^2} = 27.91 \text{ in.}$$

The resultant torque of the main and reverse springs must be equal and opposite to the bending moment of the contact pressure, the weight and the wind pressure on the bow and arms, or:

$$(15 + 2.75) \times 27.91 + 3.38 \times 11 = 532.7 \text{ in-lb.}$$

When the radial arms are lowered, the torque of the main springs (pushing them up) increases, that of the reverse springs (pushing them down) decreases. The contact pressure, which is the main force balancing the torque, therefore increases with the lowering of the

arms. To avoid interference of the contact wire and the main springs, the bow pins must be at least 3 in. above the center of the shaft. The downward range of motion from the above position, therefore, is 8 in. It has been found by trial that this range is adequate if the contact pressure increases 30 lb. while the bow moves through the whole of it.

To prove the adequacy of this range the writer will choose the springs so that the contact pressure does increase 30 lb. while the bow turns down to 3 in. above the center of the shaft, and will then calculate the actual relative motion of the bow and shaft when the bow advances from the suspender toward the other end of a span, and will thereby prove that the bow does not move beyond the available range.

In the lowest position of the bow, its horizontal distance from the shaft is $\sqrt{30^2 - 3^2} = 29.85$ in. The torque of the springs must balance the moment of the weight, the contact pressure, and the wind pressure around the shaft; or,

$$\text{torque} = (45 + 2.75)\ 29.85 + 3.38 \times 3 = 1\,435.5.$$

The increase in the torque of the springs during this motion, therefore, is:

$$1\,435.5 - 532.7 = 902.8 \text{ in-lb.}$$

The angle, a, of the radial arms (from shaft to hinges) with the horizontal, is given by sin. $a = \frac{3}{30}$, or $a = 5° \ 44.36'$. The angle, b, for the first position, is given by sin. $b = \frac{11}{30}$, or $b = 21° \ 31.63'$, and $b - a = 15° \ 47.27'$. The length of the corresponding arc described by the bow is 8.266 in. The reverse springs will be adjusted so that they act through only the first quarter of this arc and are equally stiff with the main springs. The change in torque through the first quarter of the motion, therefore, is twice as much as in any succeeding quarter. The total change in torque, therefore, is as much as in five-fourths of an arc of 8.266 in. with only one spring, or in 10.332 in.

The change in torque per inch of arc is 902.8 : 10.332 = 87.38 in-lb. This applies to the last three-quarters of the motion. During the first quarter the change in torque per inch of arc is 174.76 in-lb. From this the size and length of the springs will be determined, in the following:

The lowest position of the sliding bow will evidently be reached when the radial arms are pointing backward and are turned down by a strong head wind. Assuming the wind velocity 80 and the train velocity 50, their relative velocity is 130 miles an hour. This gives a wind pressure of 45.6 lb. per sq. ft., or, on the sliding bow, 22.8 lb., its area, together with the mean equivalent area of the radial arms,

being ½ sq. ft. By trial equations between torques and moments, it is found that the radial arms are lowered 1.05 in. below the position corresponding to 15 lb. contact pressure without head wind. With this head wind, the bow, when it passes the suspender, is only 9.95 in. above the shaft. Its downward range of motion relative to the shaft, therefore, is only 6.95 in. When the bow is 9.95 in. above the shaft, the angle of the radial arms with the horizontal is 19° 22.2′. The available range of downward motion, measured in degrees, is 19° 22.2′ — 5° 44.36′ = 13° 37.84′. The corresponding arc of the bow measures 7.137 in. Of this arc, 0.937 in. is under both springs with double torque, the remainder, or 6.2 in., under the main spring only. The contact pressure of 15 lb. exists now when the bow is only 9.95 in. above the shaft. The contact pressure in the lowest position is found from the equation between the torque and the moment of the acting forces. The equation is:

$$(2.75 + X)\ 29.85 + 22.8 \times 3 = 1\,435.5$$

where X is the contact pressure. From this, $X = 43.03$ lb. The increase in contact pressure during the down motion of 6.95 in., therefore, is 43.03 — 15 = 28.03 lb. Of this motion, 0.89 in. is under the reverse springs, and 6.06 in. under the main springs only. Counting the first part double, we have 7.84 in. Dividing 28.03 by 7.84 we obtain, where only the main springs act, 3.58 lb. as the change in the contact pressure per inch of vertical relative motion of bow and shaft, and, where the reverse springs also act, twice as much, or 7.16 lb.

The actual relative motion of bow and shaft, after the bow passes a suspender, will now be calculated. Before the bow reached the suspender, it and the shaft were moving up together along the contact wire. It is necessary to ascertain the upward velocity of the shaft at the moment when the suspender is passed. The most unfavorable case is that with maximum deflection of the contact wire. For this case the deflection is 4 ft. and the span is 220 ft. The terminal down slope of the contact wire, therefore, is $\frac{8}{110} = 7.273$ per cent. The weight of a half span of the wire is 55.8 lb. This terminal slope is reduced by the presence of a sliding bow in the span. This sliding bow lifts the wire. The suspenders are carried either by brackets or a span wire construction between pairs of poles. Their resistance to a small longitudinal motion of the suspenders is small. The tension in the contact wire, therefore, will be but little reduced by the presence of a sliding bow in a span. It will be assumed, for the present, to be constant. The contact wire carries 55.8 lb. to the suspender; when a sliding bow with a contact pressure of 15 lb. approaches, this end shear or vertical component of the tension in the wire is reduced to 55.8 — 15 = 40.8 lb. The end slope of the wire is reduced to

$\frac{40.5}{55.8} \times 0.07273 = 0.05318$. The train velocity is 73.33 ft. The velocity of upward motion of the shaft at the suspender, therefore, is $0.05318 \times 73.33 = 3.896$. The sliding bow and the radial arms are turned down by the suspender; the contact pressure increases; the upward motion of the shaft, therefore, becomes a retarded motion. The different parts of the frame have a different upward velocity and the retardation produced in them is also different. The inertia of the whole frame (exclusive of bow and radial tubes, but inclusive of their hubs) was calculated from a design and found to be equal to that of a weight of 16.1 lb. moving with the vertical velocity of the shaft carrying the radial arms. This estimate is based on the use of nickel or vanadium steel tubing of not more than the thickness needed for ample strength. A vertical force of 1 lb., acting on the frame, therefore, will produce an acceleration of the shaft of 2 ft. per sec.

Considering now what happens after the sliding bow passes the suspender, it is evident that the bow and the radial arms are turned down by the suspender and move along the wire, which latter is raised by the contact pressure. The frame and the shaft continue to rise by their inertia. The initial upward velocity of the shaft is 3.896 ft. Taking an interval of $\frac{1}{40}$ sec., the contact pressure at the start is 15 lb.; it increases during this time by an amount depending on the relative motion of shaft and bow. The retardation of the shaft depends on this increase of the contact pressure. The position of the sliding bow at the end of $\frac{1}{40}$ sec. depends on the contact pressure then existing. A formula might be developed for finding the relative vertical motion of shaft and bow, but a more convenient and shorter method is to assume the relative motion, to find the corresponding contact pressure at the end of the time interval, and to calculate from it the consequent motion of shaft and bow and their relative motion. If the result does not agree with the assumption made, another relative motion must be assumed, and the calculation repeated. After two calculations the proper value can generally be found closely by interpolation. This, though circuitous, is believed to be shorter than the development of formulas for the many different cases which must be considered. Assuming, then, the relative motion of shaft and bow during the first $\frac{1}{40}$ sec. to be 2 in., the contact pressure during the first 0.89 in. of this relative motion increases 7.16 lb. per in.; after this it increases 3.58 lb. per in.; at the start it is 15 lb. At the end of the time interval it is $[15 + (3.58 \times 0.89)] + (3.58 \times 2) = (18.19 + 7.16) = 25.35$ lb. The average contact pressure during the interval is $\frac{15 + 25.35}{2} = 20.175$ lb. This is 5.175 lb. in excess of the 15 lb. which would allow uniform upward motion of the frame. The average retardation of the motion of the shaft produced thereby is

10.35 ft. The consequent reduction of velocity during $\frac{1}{40}$ sec. is 0.259 ft. The end velocity of the shaft is 3.896 — 0.259 = 3.637 ft. The average velocity during this interval is 3.896 — 0.130 = 3.766 ft. The upward motion of the shaft, in inches, is $\frac{3.766 \times 12}{40} = 1.130$ in. The contact pressure at the end of the interval is 25.35 lb. The sliding bow has advanced from the center of the suspender by $\frac{73.33}{40} =$ 1.8333 ft. To obtain a convenient formula for finding the position of the sliding bow, with a contact pressure, P, after n intervals of $\frac{1}{40}$ sec., the distance of the sliding bow from the near end of the span is $n \times 1.8333$ ft. Of the pressure, P, there goes to the near end of the span, $P\frac{(220 - 1.8333\,n)}{220}$. The weight of the assumed wire is 0.507 lb. per ft. One-half the span (or 0.507×110) is carried to each end. The load carried to the near end of the span is

$$0.507 \times 110 - P\frac{P \times 1.8333\,n}{220}.$$

This is the shear at the end of the span, or the vertical component of the tension in the wire. The wire between the sliding bow and the near end of the span hangs in a curve. The chord of this curve is parallel to the tangent in its center. The direction of this tangent depends on the shear at this point and the tension in the wire. This shear is equal to the end shear less $\frac{n \times 1.8333 \times 0.507}{2}$. Subtracting this from the end shear and reducing the result to convenient form, we obtain $(55.8 - P)\left(1 - \frac{1.8333\,n}{220}\right)$ lb. The down slope of this tangent is less than the terminal slope of 0.07273 in the ratio of this shear to the shear of 55.8 lb. Therefore, it is

$$\frac{(55.8 - P)\left(1 - \frac{1.8333\,n}{220}\right)}{55.8} \times 0.07273.$$

The drop from the suspender to the contact point is equal to the slope multiplied by the distance, or, after reduction and expression in inches, $(55.8 - P)(1 - 0.008333\,n)\,0.0287\,n$. From this formula we get the drop of the bow after $\frac{1}{40}$ sec. for the assumed relative motion of shaft and bow of 2 in. by making $P = 25.35$ and $n = 1$. This gives for the drop of the bow 0.867 in. We found that the shaft rises 1.130 in. in the same time interval. This gives 1.997 in. for the relative motion of bow and shaft in the first $\frac{1}{40}$ sec. This is so close to the assumed 2 in. that no further trials are necessary.

For the second $\frac{1}{40}$ sec. the relative motion of shaft and bow is assumed to be 1.5 in. For the relative motion during $\frac{1}{20}$ sec., this

gives 1.997 + 1.5 = 3.497 in. The contact pressure at the end of this interval is 18.19 + (3.497 × 3.58) = 30.72 lb. The average contact pressure during the second $\frac{1}{40}$ sec. is 28.04 lb. This is 13.04 lb. in excess of the pressure which would allow a uniform rise of the shaft. This produces an average retardation of 26.08 ft., and a reduction of velocity in the interval of 0.652 ft. The velocity at the end of the previous interval was 3.637 ft. The upward velocity of the shaft at the end of the second $\frac{1}{40}$ sec. is 3.637 — 0.652 = 2.985 ft. The average velocity during the interval is 3.637 — 0.326 = 3.311 ft. The consequent upward motion of the shaft is 0.993 in. The previous upward motion of the shaft was 1.130 in. The total upward motion of the shaft during $\frac{1}{20}$ sec., therefore, is 2.123 in. From the above formula the downward motion of the bow is found by making $P = 30.72$ and $n = 2$. The downward motion of the bow during $\frac{1}{20}$ sec. is 1.416 in. The relative motion of shaft and bow during $\frac{1}{20}$ sec., therefore, is 2.123 + 1.416 = 3.539 in. A repetition of the calculation with a slightly larger assumption for the relative motion gives approximately:

Motion of shaft in $\frac{1}{20}$ sec....................	2.123 in.
Motion of bow............................	1.407 "
Total relative motion in $\frac{1}{20}$ sec...............	3.530 in.

The calculation, continued in the same manner, shows that the relative motion of shaft and bow reaches its maximum after 0.15 sec., and is then 5.99 in. After this the radial arms begin to move up again and they reach their original slope corresponding to a relative motion 0 after 0.47 sec. The arms rise further until the vertical distance between bow and shaft is 1.626 in. more than at the start. The contact pressure is then only 3.17 lb. This occurs 0.70 sec. after passing the suspender. The arms begin then to move down a second time. After a few swings the oscillation is stopped by the friction. This calculation is not accurate mainly because the tension in the contact wire, which was assumed constant, is slightly reduced by the action of the sliding bow. This reduction, however, is small, partly because the sliding bow during the first oscillation, which is the largest, is near the end of the span, and partly because the suspenders can move easily a fraction of an inch in the direction of the track. The margin for the downward motion is about 1 in.; this will be reduced by the reduction in tension in the contact wire.

It is evident that the amount of the initial upward velocity of the shaft when the bow passes the suspender is a very important factor deciding whether the range of relative vertical motion of bow and shaft allowed is adequate. This initial velocity depends on the terminal slope of the wire, the ratio between static contact pressure and the weight of a half span of wire, and especially on the train velocity.

When the radial arms are turned forward, a head wind of 80 and a train speed of 50 miles an hour cause a rise of the bow. With 15 lb. contact pressure, the bow rises to an elevation of 12.8 in. above the shaft. It will rise approximately 1.9 in. more during the oscillation. There is plenty of upward range of motion to allow this rise.

For obtaining a small equivalent weight for the bow and radial arms, it is important that the radial arms form a small angle with the horizontal. When the bow follows a curve in the contact wire, an approximately vertical acceleration of $\frac{v^2}{r}$ must be given to it by an external force, where v is the train velocity and r is the radius of curvature of the motion of the bow. This acceleration is produced by the turning of the radial arms. This turning moves the bow normal to the arms. The vertical component of its motion is only equal to the motion multiplied by the cosine of the angle, a, of the radial arms with the horizontal. The acceleration of the bow and of the ends of the radial arms, therefore, must be equal to $\frac{v^2}{r \cos. a}$. The variation, P, of the approximately vertical contact pressure produces this acceleration; this pressure forms an angle, a, with the acceleration, and its component in the direction of the acceleration is $P \cos. a$. The force to produce the acceleration is equal to the mass to be moved multiplied by the acceleration, or

$$P \cos. a = M \frac{v^2}{r \cos. a}, \text{ or } P = M \frac{v^2}{r \cos.^2 a}.$$

M is the mass which if placed at the bow offers the same resistance to turning as the bow and the radial arms. M is approximately equal to the mass of the bow plus one-third of the mass of the arms. It is evident that P, or the variation of contact pressure due to centrifugal force, is greatly increased if the angle, a, is large. To obtain a small variation of contact pressure due to centrifugal force, or, in other words, a small equivalent weight of the sliding bow, the arms should form a small angle with the horizontal. This is the main reason why the reverse springs, which greatly reduce the variation of elevation of the bow above the shaft, are necessary for obtaining a small equivalent weight. The angle of the radial arms with the horizontal when the sliding bow passes the suspender is the only important one in calculating the equivalent weight of the sliding bow. The angle, a, with a head wind of 80, and a train velocity of 50, miles per hour with the radial arms turned forward, and, when the bow passes the suspender, is given by $\sin. a = \frac{12.8}{30} = 0.4267$. This gives $\cos.^2 a = 0.8179$, and $\frac{1}{\cos.^2 a} = 1.223$. The equivalent weight, Mg, for the bow

and radial arms is 2.4 lb. This is increased, due to their slope when at the suspenders, to $2.4 \times 1.223 = 2.94$ lb. Allowing for some inaccuracy, it may be 3 lb.

Referring to the main springs: There is in each main spring a maximum torque of 718 in-lb. Their strength is given by the equation, $718 = \frac{\pi d^3}{32} s$, where d is the diameter and s the strain per square inch. Taking s, for hardened spring steel, $= 100\,000$ lb., then $d = 0.418$ in. The length of the spring is found by the equation, $f = \frac{64\ P\ l\ r^2}{\pi\ E\ d^4}$. In this equation, r is 30 in., f is the arc through which the bow moves by the application of the force, P, acting along the tangent with the radius, r. It has been found that, for $f = 1$ in., $P \times r$ must be 87.38 in. This gives for l, the length of the spring, 33.2 in. In the same manner, the reverse spring, shown on Plate II, was calculated.

Higher speeds than 50 miles an hour, together with a head wind of 80 miles an hour, are only practicable with trains of cars. Such trains have more than one sliding bow. The wire, therefore, is lifted much more than with one bow. The initial upward velocity of the shaft when the sliding bow passes the suspender is thereby much reduced, and the same available range of downward motion will generally be adequate. On roads having high train speeds, substantial steel poles will mostly be wanted; with such it is always advantageous to use strain adjusters which permit longer spans or smaller deflections of the contact wire. Either of these diminish the vertical velocity of the shaft when the sliding bow passes the suspender, and the assumed range of relative motion of shaft and bow becomes adequate in spite of the higher speed. The higher speed also requires longer radii of vertical curvature of the contact wire, for the purpose of avoiding jumping of the sliding bow. These longer radii become practicable only if strain adjusters are used.

The sliding bow here described is also suitable for the catenary suspension. Though the wire appears to be straight where this suspension is used, it is bent up by the contact pressure, especially if the suspenders are near together. With a heavy sliding bow, and a slack wire at high temperature, the curvature of the motion of the sliding bow is sharp just before a suspender is reached, and the sliding bow jumps at high speed. A light sliding bow will evidently produce less centrifugal force and less variation of contact pressure, and consequently less jumping. It will run smoothly at much higher speeds than a heavy one.

There are inevitably vertical curves and changes of direction of the contact wire at low overhead crossings and tunnel entrances. These

changes will cause much less trouble with a light sliding bow. With catenary suspension and short spans between poles, a sliding bow 4 ft. long, with radial arms 2 ft. long, can often be used. Such a sliding bow can be designed for an equivalent weight of 2 lb. Such a bow will run smoothly at very high speeds.

With the more uniform contact pressure of a light bow, a much smaller static contact pressure can be used. The result will be less wear and greater durability of bow and wire. The increase in the first cost of a diamond-frame bow caused by the modification here proposed will be in all cases amply repaid by the reduced cost of construction and maintenance of the contact line.

DISCUSSION.

R. D. Coombs, M. Am. Soc. C. E.—The speaker is of the opinion that interruptions in service caused by lateral displacement are improbable on either the 240-ft. spans with a sag of 2½ ft., as used by Mr. Mayer, or on 300-ft. catenary spans having a sag of 6 ft. Mr. Coombs.

Based on the fact that the hot and cold sags in the catenary, and therefore in the trolley wires, are approximately equal for a total variation in temperature of 140°, the tension in the trolley wire is given by Mr. Mayer as about 26 000 lb. per sq. in. Assuming that the catenaries are erected with the normal tension at normal temperature, it would seem that the increased tension in the trolley wire should be merely that due to a rise or fall in temperature of half the total variation.

The speaker is not familiar with the details of the automatic adjustment of the trolley wire used in the Blankenese-Ohlsdorf line, or other foreign lines equipped with the secondary catenary, but thinks it should not be necessary to run the comparatively inflexible trolley wire over the adjusting pulleys, as this might be avoided by attaching a flexible wire which would permit the use of pulleys of moderate diameter.

The elastic limit, of from 40 000 to 45 000 lb., assumed for trolley wire having an ultimate strength of from 50 000 to 60 000 lb. per sq. in. seems to be rather high, and the maximum stress of about 26 000 lb. per sq. in. in the trolley wire under dead load, plus 6.0 lb. wind pressure, plus bending of the wire, does not give the contact wire in the saddle suspension the same factor of safety as that used in designing first-class railroad bridges.

A parallel condition to that of many railroad bridges would be dead load, plus ice ½ in. thick, plus 8 lb. per sq. ft. for wind pressure, plus lateral bending; the total stress from which would exceed 26 000 lb.

Assuming that the single catenary and the trolley wire supported by it can be designed with suitable factors of safety, and constructed so as to give satisfactory operation, the extra expense of the messenger wire and hangers may be justified as a safeguard to prevent falling wires and the troubles incident to them.

Charles Rufus Harte, M. Am. Soc. C. E.—Mr. Mayer has developed a very interesting construction which it is to be hoped may have a practical trial in the near future. At the same time, it should be noted that the excessive stresses feared by Mr. Mayer do not always develop in the older forms of suspension. Undoubtedly, on long level tangents, with heavy anchoring, there would be heavy stresses at low temperatures, if the trolley had been well pulled up in warm weather, but, as a matter of fact, grade changes and curves offer relief, and Mr. Harte.

Mr. Harte. trolley pulled to a tension of 2 200 lb. in summer apparently does not materially increase this stress in winter under usual conditions, owing to the yielding of supports. Where trolley is hung slack, however, the changes of length are chiefly taken up in the sag, and here an adjuster may be desirable; but, certainly in New England, such a device must be automatic or else receive constant attention in order to meet the rapid and large changes of temperature of that climate.

A system of counterweights offers the ideal method of securing uniform tension, and there is little difficulty in arranging bell-cranks or of splicing into the trolley a section of steel strand, if the trolley itself is too stiff to lead direct to the counterweights. It must be apparent, however, to anyone familiar with trolley wire, that Mr. Mayer's figures and practical conditions do not agree.

To determine roughly the flexibility of 0000 B. & S. gauge hard-drawn, grooved copper trolley wire, the speaker arranged a crude form of the "Atwood's machine," of physics, the trolley wire forming the cord, and the head sheaves of a dumb-waiter the wheel. (Plate III.) Balanced weights were hung from the wire, and then one side was loaded until motion occurred. No correction was made for the considerable friction of the wheels used.

With a pulley 55 in. in diameter at the root of the groove: 127 lb. on each side required 13 lb. additional on one side to move; 232 lb. on each side required 18 lb. additional on one side to move; and, 792 lb. on each side required 64 lb. additional on one side to move.

With a wheel 33 in. in diameter: 232 lb. on each side required 33 lb. additional on one side to move.

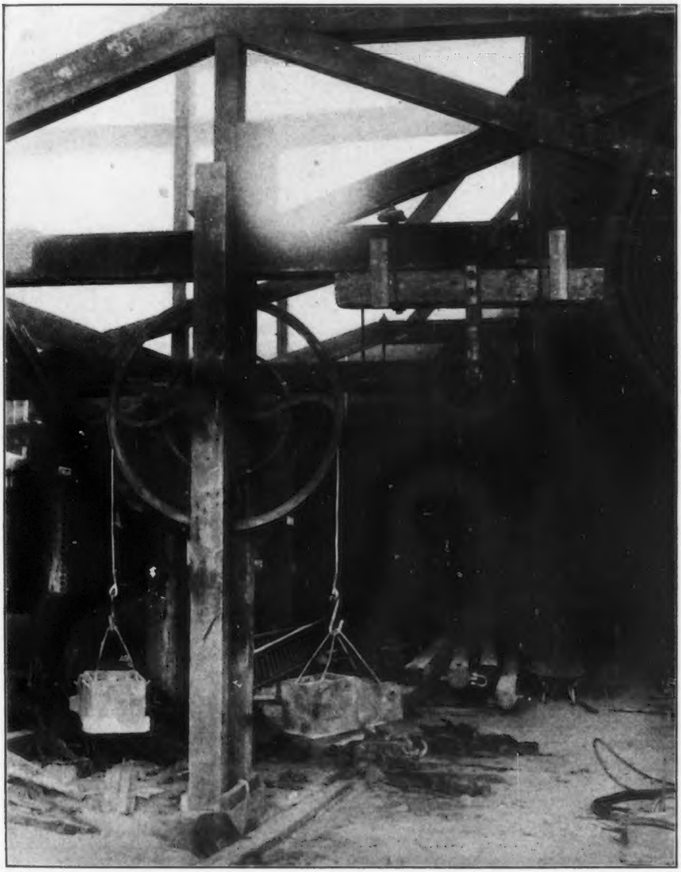
With a wheel 17 in. in diameter: 127 lb. on each side required 83 lb. additional on one side to move; and, 324 lb. on each side required 155 lb. additional on one side to move.

The Blankenese-Ohlsdorf trolley is described* as a grooved wire, having a gauge practically equivalent to 0000; Mr. Mayer gives the area as 100 sq. mm., which is 3½% less than the area of the wire used in the foregoing rough test. It would appear, therefore, that with wheels 5 ft. in diameter the trolley itself can be taken to the counterweight and a stress of 1 000 lb. can be maintained by a weight of not more than 11 000 lb., while, with a wheel 3 ft. in diameter, the weight, to secure a tension of 1 000 lb., will not exceed 1 200 lb.

For assistance in the tests, the speaker is greatly indebted to his assistant, Mr. John F. Trumbull, and to Messrs. C. W. Blakeslee and Sons, Contractors, of New Haven.

Mr. Mayer. JOSEPH MAYER, M. AM. SOC. C. E. (by letter).—The history of all construction begins naturally with practice based on feelings arising from intimate and varied contact and experience with the structural materials. It is followed, through long periods, by observation of

* *Street Railway Journal*, April 6, 1907.



IMPROVISED "ATWOOD'S MACHINE" FOR TESTING THE STIFFNESS OF TROLLEY WIRE.

successes and failures and by gradual improvements suggested by them. Analysis of the causes of failure soon leads to crude theories considering the principal forces acting on the structures and establishing principles for guidance in future design. As long as only some of the acting forces are considered, the theory must be assisted continually by observing the results of past practice. Mr. Mayer.

After all the incident forces are closely estimated and the resulting strains in the materials used are carefully calculated and compared with their safe strength, theory becomes a reliable guide for making radically new designs. It teaches the distinction between necessary and superfluous parts, and generally leads to a simplification of the structure, and, with it, to a practical certainty of success.

This process of development is substantially completed, in the design of steel bridges. The design of contact wire suspensions, on the whole, is yet in the empirical stage. The tensions in the wire are beginning to be considered theoretically, while the bending strains, which are in many designs of much larger amount, are only mentioned occasionally. The writer, thus far, he believes, is alone in having published the calculation of the bending strains in overhead contact wires.

Experience has shown that, for moderate car speeds, a wire carried by short and rigid clamps at distances of about 100 ft. gives a fairly satisfactory structure; but, with high speeds, it is found that the current collector strikes the clamps violently, and the wire when leaving them, especially in summer when the deflections of the wire are considerable. This experience has led to the adoption of catenary construction. The use of higher voltages and the consequent call for greater safety of suspension have been contributory causes. With catenary construction, the wire hangs approximately straight, but the passing collector pushes it up between the frequent suspenders and moves, not in a straight line, but in a sinuous curve.

With the present usual sliding bows, a minimum wire tension of about 2 000 lb. is now frequent practice on interurban roads, and 4 000 lb. or more is adopted with the highest speeds thus far attempted. Smaller tensions cause jumping of the collector at the suspenders, especially if these are rigid and heavy. If they are light and flexible, less trouble is experienced, but should the wire break and the collector rise, it would probably knock off the suspenders, and the wire would fall. That the foregoing minimum wire tensions are moderate is confirmed by the reported recent adoption on a prominent line of a 7/0 alloy wire of 50% conductivity and 50% greater strength than obtainable with hard-drawn copper, for the purpose of increasing the least tension to several times the largest amount above given. A 4/0 copper wire previously used did not give continuous contact with a safe amount of tension.

Mr. Mayer. With a 4/0 wire the above tensions of 2 000 and 4 000 lb. amount to 12 000 and 24 000 lb. per sq. in. A drop in temperature of 140° increases them, with constant length of the wire, by 21 000 lb. per sq. in. It is claimed that the length of the wire is changed considerably by the presence of curves and the yielding of the anchorages and poles. The change in the length of the contact wire with constant tension due to 140° change of temperature is 7 ft. to the mile. If the tension is to remain constant, the length of the wire must change by this amount. Very little calculation shows that even on lines of much curvature the actual change in the length of the wire due to change of temperature is but a small fraction of this.

The increase in the tension of the wire with drop in temperature, therefore, is real and not imaginary. In so far as the poles yield at curves and thereby shorten the wire in winter, the latter is made to move toward the center of the tangents. The suspenders and steady strains become inclined against it, and the wire is bent into wave lines which increase the jumping of the current collector and produce large bending strains, thus more than neutralizing the slight reduction in the tension. To reduce this trouble, frequent readjustment of the suspenders and steady strains becomes necessary, and the cost of maintenance is increased.

Against the writer's theory of the sliding bow, the objection is raised that the irregular movement of the car roof carrying its frame was not considered. The writer's design connects a light flopper and an aluminum bow, by springs of very moderate inertia, to the main supporting frame. Any vibrations below the springs are absorbed by them and cause but trifling variations of the contact pressure.

Under the suspender described in the paper, the collector moves at low temperature in a curve having two parts convex below, and a central part convex above, as shown by Fig. 1. At high speeds, to prevent

FIG. 1.

interruption of contact along the curves convex below, the weight of the collector and of its rigidly attached parts must be small and the radius of the curves large. To make this possible, strain adjusters are indispensable at high speeds. This is a serious defect of the suspenders described.

Another defect is their lack of adaptation to trolley wheels. In endeavoring to remove these defects, the writer invented the hanger shown in Fig. 2. It consists of a flexible tapering bar with such cross-sections that it will bend vertically and horizontally approximately in circles. The wire is attached to it by clips, and is supported be-

Mr. Mayer.

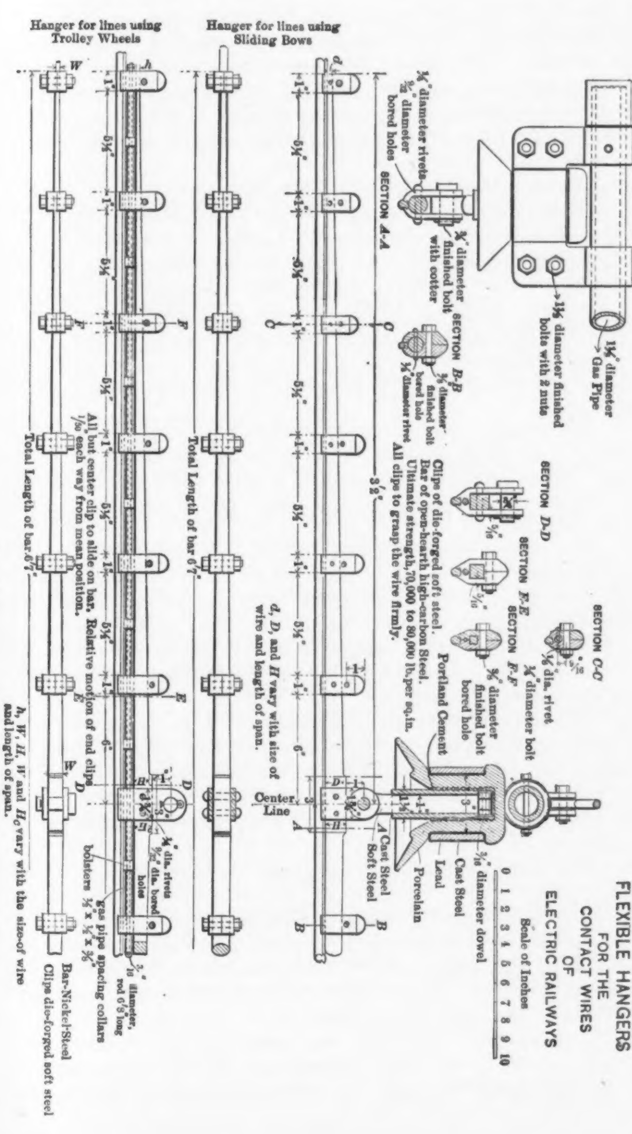


FIG. 2.

Mr. Mayer. tween the clips by bolsters which are held in position by a steel rod and tubular collars. Under this hanger the attached wire is always convex upward. The contact pressure is here slightly increased by centrifugal force, and no interruption of contact will occur, even with heavy current collectors at the highest speeds. If the bar is made of nickel-steel, very large deflections of the wire can be used and a great factor of safety can thereby be obtained.

The bending strains in the wire are only about half as great as those with the suspender described previously. With spans of 240 ft., without strain adjusters, and of 300 ft., with strain adjusters, the maximum tensions in the wire, with suitable deflections, and the amounts of ice, wind pressure, and changes of temperature previously given, are about 17 000 to 18 000, and the total maximum strain is less than 24 000 lb. per sq. in. Greater factors of safety can be obtained with shorter spans.

The advantages of the flexible hangers may be summed up as follows:

1st.—The bending strains in the contact wire are reduced by the avoidance of all short bending, the tensions by the use of liberal deflections. Spans from one and one-half to three times as long as with other suspensions, therefore, may be used, and a greater factor of safety is obtained.

2d.—The current collector moves in curves of large radii, whereby, even with great train speeds, a steady contact is obtained, and the contact pressure is never made excessive by centrifugal force. The average contact pressure can be made very low, and the hammering of the wire at the ends of the hangers is entirely avoided; the durability of the current collector and of the wire is thereby increased, and their cost of maintenance reduced.

3d.—The sight of the signals is interfered with on curves by the many poles of the ordinary suspensions, and especially by the ropes and numerous hangers of the catenary suspensions. The first evil is greatly reduced, and the latter avoided, by the flexible hangers.

4th.—Since the tensions in the wire are always very moderate, no readjustments are ever required.

5th.—The same hangers will also answer on curves and at changes of grade.

6th.—As compared with catenary suspensions, the load and wind pressure on the carrying structure is only from one-fifth to one-third.

7th.—With high speeds, all other suspensions require a large wire for strength; with the flexible hangers a No. 0 wire is safe, and can be used wherever its conductivity is adequate.

8th.—The apparent safety of the usual catenary suspensions is often illusory. With the large working strains in the wire, inevitable with them in severe climates, it will stretch during the first cold

winter and will need readjustment in the spring; after a few years, Mr. Mayer. and repeated stretching, it will break occasionally, then the sliding bow will rise, knock off the hangers and bring it down.

With the moderate working strains adopted with the flexible hangers, the wire can be tested during erection by temporarily exposing it to a tension 50% larger than the maximum working strain. It will then be safe thereafter.

In concluding, the writer wishes to point out the principal difference between his method of investigation and that followed by some others. The writer's method is apparently theoretical; that of many others is exclusively what they call practical, mainly by means of experiments and tests.

The writer claims that the suitability and value of a contact wire suspension or of any other bridge cannot be practically ascertained by short-time tests. With railroad bridges, tests have been generally abandoned; they have been replaced by scientific design. The railroads prescribe the strength of the materials to be used, the unit strains, and the loads to be assumed. They call for complete strain sheets. If the loads and unit strains are well chosen, all the calculations for details and main members correctly made, and the materials and manner of execution closely inspected, a safe structure is obtained which is used without further preliminary tests.

The wind pressures, changes of temperature, and ice loads, which are so important in any contact wire suspension, cannot be provided for a test. Even if this were possible, a test would only show that the structure could withstand once the combination of conditions chosen. It would not give the all-important factor of safety. The only way to ascertain the degree of safety and the value of any such structure is to calculate the strains produced in all its parts by the practically occurring combinations of conditions, and compare them with the strength of the materials used and thereby find the factor of safety.

By looking at a structure while it is in operation, but little can be ascertained in regard to its strength and durability, beyond the fact that it is not actually falling down at the time. The call for tests on similar structures, therefore, is not a very scientific demand, and it is not practical, though it has the appearance of being so. It arises mainly from laymen and from engineers whose principal work is with machinery of a much more complicated nature, where tests are a necessity to supplement the calculations. In early stages of structural design and with structures too complicated for complete calculation, tests may be the only available means to obtain information on the practicability of a structure. The value of such complicated structures, however, can only be ascertained by years of trial. A complete analysis of all the forces acting on the structure, and of the strains

Mr. Mayer. produced by them, is the only means by which an engineer can obtain a sufficiently thorough knowledge to enable him to predict the success or failure of a simple structure; and it is also the only method by which a really scientific and economical design can be made; it is the method which has alone proved successful in the design of steel bridges, which most closely approach all contact wire suspensions in their essential natures.

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TRANSACTIONS

Paper No. 1078

THE USE OF REINFORCED CONCRETE IN ENGINEERING STRUCTURES.

An Informal Discussion Presented at the Meeting of January 8th, 1908.

By Messrs. E. P. Goodrich, Edwin Thacher, Sanford E. Thompson, William H. Burr, T. Kennard Thomson, D. W. Krellwitz, Guy B. Waite, C. L. Slocum, Myron S. Falk, Rudolph P. Miller, Eugene W. Stern, H. C. Turner, and E. P. Goodrich.

E. P. Goodrich, M. Am. Soc. C. E.—The use of reinforced concrete in engineering structures has had a phenomenal development, both as to the amount built in each succeeding year and as to the variety of applications made. Its field of usefulness is rapidly broadening, and its exploitation is believed to have been overdone in a few lines. Mr. Goodrich.

The theory concerning the mode of action of the two materials involved, is constantly undergoing modification, making it more perfect through deduction from experiment. This is the scientific method of development of any art; and in this particular branch of the building art, it is believed that by experiment alone can proper working stresses be determined, upon which to base all designs of structures. Such stresses should be deduced primarily from fatigue experiments, and not be chosen as arbitrary fractions of ultimate strengths. It is believed, further, that every careful designer should take proper account of the secondary stresses induced in structures like buildings and arches, by the increasing permanent set caused by repeated loading.

Mr. Goodrich. Experimental research is yet much needed along several different lines connected with this subject. More light is desired as to the cause and cure of the retrogression in the tensile strength of cement briquettes, often disclosed. More extended compression tests are also needed to determine the presence and amount of any retrogression in the compressive strength of concrete. When tests, published in engineering periodicals, show, at the end of 2 years, values hardly greater than those at the end of 7 days, it would seem as if this subject needed most careful investigation.

If possible, a cement of higher compressive strength should be developed, especially for use in concrete columns. Perhaps this is impossible with the materials involved, because of their very nature, and because the strength of the aggregate has been practically reached; but unless some considerable increase can be secured, it would seem as if concrete columns would have to be excluded from consideration in high building design; that is, in structures higher than perhaps six stories. In their stead, structural steel columns would seem necessary, but they should be heavily fire-proofed and entirely filled with a concrete of cheap quality. It may seem excessive to some engineers, but it is believed that experience has shown the necessity of fully 3 in. of good concrete fire-proofing over all extreme edges of such columns.

The best design for the steelwork of columns of this variety, is believed to be of angles latticed or battened, of channels similarly fabricated, or perhaps wide-flanged I-beams, or the usual Z-bar types.

Columns of the Considère variety are believed to be proper, if a suitable relation exists between the spiral and the longitudinal reinforcement, and if a sufficient quantity of each is used. In such columns, a lower limit should be set on the quantity of each kind of reinforcement, and an upper limit on the size of the opening between the parts. It may not be out of place to state that the inventor of spiral reinforcement himself uses spirals of very thick material with a comparatively small pitch, and it is believed that a large majority of the columns being erected at the present time, and considered of high carrying capacity, would not disclose any appreciable excess of carrying power if tested to failure.

In reinforced concrete columns, with longitudinal rods as the principal reinforcement, an upper limit should be set on the percentage which may be allowed. In addition to the fact that many laboratory tests show lower efficiencies for rods of large diameter in concrete columns, it would seem as if the use of rods more than 1½ in. in diameter, or aggregating more than 5% of the total area of the column were of more than doubtful value, simply from the impossibility of being certain that enough adhesion is developed to secure the theoretical compressive stress in the steel itself. It might seem as if more

nearly practical conditions would be secured, in laboratory tests, if all reinforcement were kept away from the ends of the concrete columns a distance equal to at least one-fourth of the diameter of the column. Mr. Goodrich.

It is believed that too little attention is given to the design of the footings under such columns, especially with regard to a proper transmission of the stress in the longitudinal rods into the foundation concrete, and that in most work not more than half the proper number of ties are used to prevent buckling in the vertical rods. The disposition of the latter, so as to prevent easy depositing of the concrete, is imperative, and those varieties of columns in which the steel is distributed very uniformly through the whole column section are viewed with distrust, however superior they may be considered from a theoretical standpoint.

Great care should also be exercised in the design of the beam and girder reinforcement, to prevent a congestion of steel in the column sections at the floor levels. Much ingenuity can profitably be expended in obviating this trouble.

It is well known that the addition of steel to increase the compressive strength of concrete columns is not on the side of economy of first cost, but only of economy of floor area occupied. It would seem, therefore, as if the best practice would be to introduce steel only to carry bending stresses, and to use a cement of higher quality (if obtainable), or a richer mixture of the commercial product, and thus secure higher working stresses with correspondingly smaller sections.

The subject of impervious concrete is of vital importance for those who are interested in the construction of dams, reservoirs, conduits, sewers, and water pipes; of hardly less interest in connection with sea walls, retaining walls, bridge abutments, and building foundations; and even of considerable interest in regard to the superstructures of buildings, arch bridges, etc. Many experiments have been made as to the perviousness of different mixtures of different aggregates of different sizes, but, apparently, something further is necessary. Great things are claimed for the several patented wet and dry compounds now on the market, designed to render concrete impervious, and, until further progress can be made in this line, the use of the best of these in all concrete work is strongly recommended. Perhaps an impervious cement will soon be evolved, produced either by the addition of one of the present products to the practically finished cement product, somewhat as gypsum is now added, or by some other substance which the inventor may work out. Such a cement is greatly to be desired, if for no other reason than to prevent the unsightly discoloration from efflorescence which now defaces almost all exterior cement work. This may be partially cured by the use of a so-called "non-staining" cement, but all those now on the

Mr. Goodrich. market are unsatisfactory, either as to effect or first cost. A better and cheaper one is essential.

Although other elements than moisture are involved in the rusting of steel, the use of an impervious cement would have gone far toward preventing the absolute disintegration of the metal backing of some stucco work examined by the speaker on several occasions, and would make one feel much safer as to the probable life of some of the reinforcement which has been plainly visible on the bottoms of floor slabs when the latter were slightly scratched with a knife or other sharp implement. It will not be very long before some of such floors will show signs of failure, especially those in which wires of small diameter, or sheet material of small thickness have been used for the reinforcement.

Impervious concrete is also of vital importance in foundations, and walls of reservoirs, conduits, etc., through which water will percolate slowly. It is known that at least one heavy retaining wall became honeycombed to such an extent that failure resulted; and in one high building on lower Broadway, in New York City, the sub-cellar walls have been screened, apparently to hide the process which may possibly be slowly producing a similar effect in that structure.

Again, the few experiments which have been made with regard to electrolysis of embedded steel in wet concrete, together with the astonishing phenomena observed in one reinforced concrete street car barn, in which hot metal was sometimes encountered when a trolley pole left the trolley wire, seem to be convincing evidence of the necessity of using impervious concrete in all reinforced foundations which may be in the line of electric earth currents.

It would also seem wise to use only such concrete in reinforced concrete piles, because they are relatively slender members, and any disintegration of either the reinforcement or the concrete in them would be of grave moment. As to the general subject of concrete piles, not enough is yet known. While a strong difference of opinion may exist, it would seem as if fewer objections could be raised against those piles which are moulded in plain sight and driven as is a wooden pile, than against those piles which are moulded in place. The latter rarely are properly reinforced, and it is extremely likely that the fresh concrete will be displaced before it has properly set, by operations in their vicinity.

A very heavy hammer should be used for driving such piles, one weighing at least as much as the pile being very important.

It is probable that impervious concrete will partially solve the problems incident to the use of cement in sea water, whether the disintegration caused by the latter be chemical or mechanical in its nature. It has already been demonstrated, at the New York Navy Yard, for example, that a rich face mixture, rendered more im-

pervious by careful surface treatment as soon as the forms are removed, is the best preventive of disintegration. It is hoped that someone, therefore, will produce a water-proof cement. Mr. Goodrich.

More care than is often taken, should be exacted with regard to the placing of reinforcement. Some hints have been given of the evils incident to poorly designed columns, placing floor reinforcement too close to the surface, and the dangers of electrolysis. All these may be obviated to a great extent by the exercise of care in design and execution. The compulsory use of reinforcement fabricated in units, in place of separate bars in beams and girders, is advocated. It is believed that the small possible saving which is claimed for the latter method is more than offset by the insurance that a rod or two will not be accidentally omitted from an important member, or a short one thrown in to take the place of a lost longer one. Such cases have been actually observed, even with the most perfectly organized forces, and one superintendent of a company which still advocates separate bars, once said the company could have some of his salary if they would use units, because of the immeasurable lessening of responsibility on his shoulders.

More attention should be paid to the subject of shear or diagonal tension in reinforced concrete beams and girders. The fact that certain empirical systems have produced many buildings which have not collapsed under load, is no proof of the adequacy of their reinforcement in this respect. It is believed that, in much work now under way, while the factor of safety against failure through tension or compression is four or more, the margin of safety, with regard to diagonal tension, is much smaller. The care taken with this point of design by foreign engineers is far above that common in America. Many more experiments, covering various ages and arrangements, are urgently needed.

The ideas incident to the use of discrete structural members are not applicable to the design of monolithic concrete structures. In the latter, the continuity of the members should be recognized, and the reinforcement of columns, beams, girders, and floors, should be arranged so as to make the parts act as rigidly connected elements. The design of columns simply as compression members, entirely ignoring the bending produced by unbalanced loads on rigidly connected girders, is not considered the best practice; and the use of the factor, $\frac{1}{8}$, in the moment formula, is an inheritance from the older methods used in timber and steel. Even the use of so much reinforcement at supports as corresponds with the factor, $\frac{1}{10}$, is believed to be entirely inadequate; and the speaker ventures the prophecy that progressive failure is taking place in many structures designed with only that quantity of steel at the points in question.

It is further advocated, that designs be made so that tests of se-

Mr. Goodrich. curity can be carried out on the very day the centering is allowed to be removed. It is also recommended that the tests be exacted on those dates. Such specifications, rigidly adhered to, would reduce to a minimum the danger and number of premature failures.

Deformed rods may be better in theory, but almost no practical proofs of their superiority have been produced, as far as known. Laboratory tests are hardly conclusive, since many experiments on beams actually seem to show some kinds of rods to be really detrimental to the best results.

More experiments are very desirable concerning the effect of the proportion of water used in the original mixture and the effects of continued and intermittent saturation of the concrete, upon the adhesion between it and smooth rods embedded therein. Perhaps the use of impervious concrete would solve this difficulty, irrespective of the actual effects produced by excessive moisture. More fatigue experiments, also, are essential to a full knowledge of this subject, and the very few so far reported along all lines are worthy of the highest commendation and the most careful study.

The compression experiments of this kind, in conjunction with those carried to rupture on columns of the Considère type, seem really to show the justice of allowing high stresses on such columns. As long as the elastic limit of the concrete is not reached, since columns reinforced in this way show very large deformation before final failure (thus reducing the danger of the latter), there would seem to be no good reason for restricting the working stress to the low figures at present usually exacted for plain concrete or longitudinally reinforced columns.

Nor do rods of high elastic limit appear to be advantageous, under ordinary conditions. Since all varieties of steel have practically the same modulus of elasticity, and since the first tension cracks in the concrete appear at approximately the same strain in all specimens, and consequently at the same stress, irrespective of how much higher the elastic limit may be, the relative amount of the latter is of no importance provided it is beyond the usual working stress.

Perhaps such rods may be of value in column work, where high stresses are used, and they may be advantageous in the reinforcement of long walls against shrinkage, but, even in these positions, the advantage is not evident. Reports as to actual structures of the last mentioned kind, where no cracks have appeared, together with the amount of steel introduced, are greatly to be desired. It is possible that the distribution of the reinforcement is also influential to some extent.

The character and size of the aggregate does not receive half the attention it deserves, and the quantity of water being used, especially in the manufacture of much cement brick, concrete blocks, and orna-

mental cement work, is entirely inadequate. Very few persons are interesting themselves in the artistic phase of the subject, and the results attained in most part are still considered rather crude. There are some beautiful exceptions to this statement, however. Mr. Goodrich.

Experiments should be made as to means of securing more uniformity of color of stucco, and the application of color to cement surfaces should receive more study. In Europe there are some beautiful examples of such work. While some progress has been made in devising effective and pleasing results in surface treatment of concrete work, there is still ample opportunity for improvement. All are familiar with the terazzo effect of good granolithic work, and many have seen surfaces which have been picked, axed, hammered, or treated with a sand blast. Some of the effects produced by washing, with heavy scrubbing while quite fresh, and of etching with weak acid are fairly pleasing, but probably the use of stucco in all its several varieties will eventually predominate. Colored tile can also be used, either in connection with stucco or in combination with selected aggregates, and treated with water or acid to bring out the color.

With the wider use of stucco, the necessity of securing a perfect bond between it and the foundation material will be more apparent. Several patented and secret processes are now in use, but none is beyond reproach, and in this there is a wide field for improvement. When eventually produced, such a bonding process should be used, even between parts of work done on succeeding days.

The engineer should pay more attention to the subject of forms. If specifications, hitherto, had not been so indefinite in regard to this item, fewer premature failures would have taken place. The practice to be followed in the erection of at least one important arch, of designing and specifying in detail all points as to the centering, can be followed profitably in lesser structures. With this element of risk removed, wherein the contractor has an opportunity to involve seriously the safety of the work by his faulty design and erection of falsework, and with the use of reinforcement in units designed by an engineer of long and wide experience, there is no reason why reinforced concrete work, eventually, should not become absolutely safe and fairly economical. Only one other point remains: the process of manufacture of concrete should be inspected as carefully as the production of structural steel and the grading of timber. Then the ideal will have been reached. Meanwhile, a careful study of the problem of forms is exceedingly profitable, because, in the cost of finished work, that of the labor and material thus involved often exceeds 40%, and sometimes approaches 75%, of the total cost; and, when carefully done, it may be reduced to 25%, where conditions are favorable. The rapid deterioration of all form material, because of wear and tear from repeated use, makes this item of cost high, even when the forms are

Mr. Goodrich. used a great many times. Doubtless, metal will eventually be used to a great extent, although wood will continue to be necessary for many parts. Staff is being used to excellent advantage, even for comparatively simple work, but it is not probable that its use will ever be very extensive. Some device which will remain a permanent part of the structure will probably be used, because these parts themselves can be moulded in shops where few forms are necessary, and the latter can be used a great number of times. In a similar way, the manufacture of structural members in a factory, by machine, or in such manner that few forms are necessary, will also be more widely developed where conditions make it possible.

In the labor element, a reduction can often be made by handling the forms in large units by derricks, and many devices are constantly being invented to do away with the costly work involved in the use of the saw, hammer, and nails. Bolts and a wrench, and work cut to length in a mill, are more nearly ideal. In all probability, less attention will soon be given to the finish of the work as it comes from the forms, because, for most classes of work, a better quality of surface finish is desirable, and more than enough money can be saved by using cruder forms, to cover the cost of such surface treatment.

Perhaps it is yet too early to discuss the subject of standardizing the sizes of beams, percentages of reinforcement, etc., but such a step will doubtless be taken just as soon as the art has outgrown its present really experimental stage.

Finally, a plea is made for more rational municipal building regulations and architects' specifications, in the framing of which engineers should have a hand. When the designing engineer and the man in charge of the furnishing of materials and erection of the work, are distinct individuals, better results will be attained; and owner, architect, engineer, and contractor will then all be striving for the most economical and artistic structure possible.

Mr. Thacher. Edwin Thacher, M. Am. Soc. C. E.—The effect of sea water upon Portland cement mortar and concrete, and upon steel embedded therein, is a subject which has received considerable study from American and foreign engineers and chemists, for several years past; but the investigations thus far made appear to have resulted in very little positive knowledge on the subject. There is considerable conflict of opinion between foreign experts themselves, and between foreign and American experts. What is most desired is to know why certain works have failed, and why other works have stood the tests of many years without any signs of decomposition or injury. When this is known it will be possible to write specifications for future work in which the chemical composition of the Portland cement used, and the mixture, manipulation, and placing of the concrete shall be such as will insure uniformly safe and satisfactory results. According to the best known

European writers on the subject, the use of Portland cement concrete in sea water is attended with great risk of chemical decomposition, and it is difficult and expensive to carry out their recommendations, in the way of precautions to be observed to overcome partially the risk of such a result, and their conclusions do not appear to be justified by experience in America during the past twenty years or more. M. Feret states that no cement has yet been found which will give absolute security against the decomposing action of sea water, that sulphuric acid is the principal cause of decomposition, that the cement should be low in alumina, and as low as possible in lime, that puzzolanic material is a valuable addition to the cement, that gypsum should be used sparingly, that fine sand used in mixing is injurious, and finally that the mortar must be such as will give a dense and impervious concrete. Mr. Thacher.

Dr. W. Michaëlis also recommends a completely impervious mixture, but differs from M. Feret in recommending that at least one-third of the sand used in mixing must be very fine. If the whole body of the concrete is not impervious, he says, this impervious layer should surround the porous kernel on all sides, and even underneath. He advises a cement rich in silica and as poor as possible in alumina and ferric oxide, also the addition of puzzolanic material to the cement.

M. Le Chatelier considers that the aluminous compounds in Portland cement are the direct cause of its disintegration in sea water, and advises that the alumina be replaced by oxide of iron. These foreign authorities do not give the chemical composition of a practical Portland cement, such as they would recommend for work in sea water, but satisfy themselves by condemning to a greater or less extent every constituent of Portland cement, except silica, and no manufacturer has yet succeeded in producing a satisfactory Portland cement containing this material only.

The writer has communicated with quite a number of American engineers who have had extensive experience in the use of concrete in sea water, and, almost without exception, the results have been highly satisfactory, notwithstanding the fact that very little precaution has been observed regarding the chemical composition of the cement, or the impermeability of the mixture; and the damage sustained has been confined mostly to points between high and low water, apparently due to mechanical causes more than to chemical decomposition. Joseph E. Kuhn, Major, Corps of Engineers, U. S. A., Norfolk, Va., is of the opinion that little apprehension of chemical action need be felt when standard and well-proved brands of seasoned cement are used. He mentions a sea wall built at Fort Monroe, just outside low water, fifteen years ago, of 1 : 4 : 8 concrete, with two-man stone incorporated. It has been exposed to wave action from storms, in which the beach sand was stirred up, and hurled against the wall with great

Mr. Thacher. force, also to tides and heavy swells from steamers. This mixture would naturally give a very porous concrete, but it is hard and tough, and no indications of chemical action or damage of any kind are noticeable except between high and low water, where the wall has in places been reduced in thickness as much as 4 in. This face has been repaired by 1 : 2½ : 4 concrete. Major Kuhn concludes that, by using a Portland cement of good quality, and a dense and strong facing layer when exposed to the action of the water, concrete-steel structures are as safe in salt as in fresh water.

C. W. Staniford, M. Am. Soc. C. E., Engineer in Chief, Department of Docks and Ferries, New York City, says:

"In the work of constructing the bulkhead or river walls around Manhattan, which has been in progress for the past 30 years, and is now being continued, no extra precautions are taken on account of the concrete being laid in sea water, except the use of first-class material and careful work."

Practically all the river wall, from low water up, has a granite face, backed by concrete in place, and heavy concrete blocks set in place with derricks from low water down, and the work is in perfect condition, after, in many cases, a period of 30 years. This applies also to concrete blocks laid above water at points not readily visible, and to concrete laid *en masse* above low water during the past 8 years, except in one location where, between low water and 2½ ft. above, the concrete shows some signs of pitting, and slight disintegration, which indicates a wear occasioned by the extreme pressure of ice during the long low-water slack.

S. W. Hoag, Jr., M. Am. Soc. C. E., Assistant Engineer, Department of Docks and Ferries, says:

"As regards chemical action, the experience in New York Harbor ought to be valuable, as our waters carry sewage probably not equalled in any smaller city. If chemical action counts for anything, I think it would in the harbor of New York along the North and East River waterfronts. I do not think that the possible deterioration from chemical action is likely to amount to much, unless the exposure is in close proximity to some chemical works. The above remarks are predicated on first-class material and workmanship."

A committee of the Association of Railway Superintendents of Bridges and Buildings made some investigation on the subject of concrete in sea water, and some of the replies to its inquiries are of interest and may be noted as follows:

A. Where there is no ice, concrete made in air with fresh water and sunk in sea water, works well. We would not deposit concrete direct into sea water. Disintegration more rapid than if deposited in blocks. Where there is large ice formation, concrete between high and low water will disintegrate from ⅛ to ¾ in. annually. Stone facing recommended.

B. Mix dry, no water, and deposit through chutes; depositing in blocks preferable; tides and frost have no appreciable effect on blocks. Mr. Thacher.

C. Concrete deposited direct into sea water gives perfectly satisfactory results if the materials and work are right. The cement should contain not more than 2% sulphuric tri-oxide. Concrete should not be leaner than 1 : 2 : 4. Stone facing preferred between high and low water.

D. A concrete pier at Warren, R. I., built about 25 years ago, of 1:3 mortar, is sound except between high and low tide, where it has worn away in places from 4 to 8 in., due to ice and tide. Current about 8 miles an hour.

The committee reports in favor of depositing concrete direct into sea water. It considers this method the cheapest and best, and is of the opinion that, with good material, properly mixed and handled, and with a granite face above low water, it will do good service.

Louis C. Sabin, M. Am. Soc. C. E., says:

"Many of the most eminent and conservative engineers consider that most failures are due to improper specifications, proportions, and manipulation, rather than to any defect in the cement."

William B. Mackenzie, Chief Engineer, Intercolonial Railway of Canada,* has used concrete in eight different places in clear sea water, and in every case disintegration has taken place between high and low tide, from ½ in. to 6 in. in depth. The concrete was generally 1 : 2 : 4. He learned that, where sea water carries sediment, the sediment penetrates into the pores and coats the surface, and no disintegration takes place.

Martin Murphy, Provincial Government Engineer, Nova Scotia,† has used concrete extensively for bridge piers since 1883. Some of the bridges were within the influence of the turbulent tides of the Bay of Fundy, most of them exposed to heavy drift ice, and all of them to extremes of temperature, yet but one failure can be recorded, and that, in his opinion, was due to careless workmanship.

J. G. Theban, Assoc. M. Am. Soc. C. E., Engineer in Charge of the Department of Bridges, Borough of the Bronx, New York City, has made an interesting experiment relating to the preservation of steel embedded in concrete in sea water. On August 24th, 1904, or somewhat more than three years ago, he sank in Pelham Bay, in 20 ft. of water, a shallow wooden box, in which ten steel Thacher bars, spaced at equal intervals, had been spiked to wooden cross-pieces. A bucket of 1:2:4 concrete was then lowered and dumped on and around these bars. After one month the box was raised and placed at low tide, where it was covered with sea water twice every 24 hours. The bars have been removed from time to time, and all have been found free

* *Engineering News*, October 31st, 1907.

† *Transactions*, Am. Soc. C. E., 1893.

Mr. Thacher. from rust. The speaker saw the last bar removed on January 1st, 1908, and it and also the spikes with which it was fastened were free from rust. Only a thin film of grout at most could find its way under the bars at points where they were in contact with the wood, but no rust could be discovered at these points.

Mr. S. E. Thompson. SANFORD E. THOMPSON, M. AM. SOC. C. E.—Columns represent the most vital part of a building, since the failure of one may cause the fall of the entire structure. The extreme variations in the fundamental assumptions in different private specifications, and also in city ordinances, make it imperative that the subject should receive more accurate and scientific treatment. As an illustration of the variety of ideas as to what constitutes safety, the extremes may be cited of certain city ordinances which permit a load not greater than 350 lb. per sq. in. on the column, and the value which is sometimes used in private practice of 1 000 lb. per sq. in. based on the entire cross-section of the column without appreciable reinforcement. The convincing argument, once addressed to the speaker by a prominent architect, for the adoption of the latter value in an important structure was that buildings in the Middle West had been designed and constructed with this unit compressive stress and were still standing.

The owners of a building frequently bring great pressure to bear upon the designer to reduce the size of the columns in the lower stories. This is not to be wondered at when it is considered that their dimensions may be 30 or 36 in. square, and thus require an appreciable amount of floor space.

It is well to recognize at the start that reinforced concrete columns, of a section which will compare favorably with steel, cannot yet be safely and economically constructed. A design after the principles followed by Professor Burr in the McGraw Building perhaps approaches a minimum section as closely as is possible, but, even here, only a low unit stress can be allowed on the steel without over-compressing the concrete. It may be laid down as a general principle that, not only is it cheaper to resist compressive stress with concrete than with steel, but also that concrete is cheaper than any combination which may be made of steel and concrete.

In order to reduce the size of concrete columns, four distinct methods have been used:

(1).—Rich proportions,
(2).—Vertical reinforcing steel,
(3).—Structural steel reinforcement,
(4).—Hooping or banding.

The use of a very rich mixture has much to commend it. The ultimate strength, by using a 1 : 1 mortar, may reach 5 000 lb. per sq. in.,* and the modulus of elasticity will also be so high that the deformation will be slight.

* "Tests of Metals," U. S. A., 1904, p. 386.

The introduction of vertical steel rods is indicated by the majority of tests* to be a satisfactory manner of increasing the strength, but the low stress which can be taken by the steel without permitting too great deformation of the concrete, makes this an expensive method, and the percentage of steel is limited, not only by economical considerations, but also because of the difficulty, especially when deformed rods are used, of placing the concrete around them properly. Mr. S. E. Thompson.

The use of structural-steel shapes for reinforcement has already been so fully considered in previous discussions that no further mention need be made here.

Hooping or banding, first introduced by Considère in France, perhaps more than any other method of reinforcement, has caught the popular eye, with a resulting tendency to great extremes of loading. For this reason, it behooves engineers to examine very carefully the underlying principles involved in this method of reinforcement and the results of experiments thus far made.

To illustrate the position taken by many conservative engineers on the subject of hooped columns, it may be worth while to study for a moment the real action which takes place under loading, as shown both by theory and tests.

When a load is placed upon the top of any column, it causes vertical compression or deformation, with, at the same time, a lateral expansion. The lateral expansion in concrete columns, as shown by tests upon plain and upon reinforced columns by Mr. J. E. Howard at the Watertown Arsenal,† and by A. N. Talbot, M. Am. Soc. C. E., at the University of Illinois,‡ is at first very small. Any stress produced in the steel hooping must be proportional to its deformation or stretching; hence, with small lateral expansion of the concrete, there can be but slight stress in the hoops. For this reason, and also because of the initial shrinkage of the concrete, which the lateral expansion must first overcome, scarcely any stress or pull comes upon the hoops until the concrete within them has reached a loading equal to the breaking load in plain concrete. As this load is approached, the modulus of elasticity of the concrete becomes very much lower, and consequently both the vertical and lateral deformations become much greater. Then, and not until then, does the lateral pressure begin to act appreciably upon the hoops. In other words, up to the very crushing strength of plain concrete, the value of the hooping is actually negligible. From then on, the reinforcement takes practically all the load, and a high ultimate strength may be attained, although coincident with great shortening of the column.

It is evident that, if concrete is confined in a tube, advantage can be taken of the added strength due to the tube. On the other hand,

* "Tests of Metals," U. S. A., 1904, p. 386; 1905, p. 377.

† "Tests of Metals," U. S. A., 1905, pp. 293-336.

‡ *Proceedings*, American Society for Testing Materials, Vol. VII, 1907, p. 382.

Mr. S. E. Thompson.

if hoops are very far apart, it is evident that the concrete, when it reaches a stress equal to the strength of plain concrete, will be thrust out between the hoops. Professor Talbot's tests,* using a gradually increasing load, indicate that, with ordinary spacing (the effect of different hoop spacing is not definitely discussed in the advance report of the tests thus far made), the hoops will effectually restrain the concrete within them. The effect of repeated and continued loading was not investigated by him.

Even with the concrete restrained within the hoops, the shell of concrete outside of them, which is necessary for fire-proofing and for the protection of the steel, begins to crack and peel at about the same load as that which will cause complete failure in unreinforced concrete. Professor Talbot, in fact, states as a general proposition that: "Cracking and peeling of the concrete appear at loads corresponding to the ultimate strength of the concrete."

This applies to hoops held rigidly. If the hooping is in short spiral sections, with the ends of the wire or rods simply lapped or insecurely fastened together, it follows, inevitably, that the spiral must give way and unwind as soon as it is exposed by the stripping of the concrete from the steel. Consequently, the breaking strength of a column hooped in this way will only be effectively equal to that of an unreinforced column.

The modulus of elasticity of the concrete within any hooping, after the point of exterior cracking is reached, drops very rapidly, reaching, in the two diagrams shown in Professor Talbot's paper, less than 300 000 lb. per sq. in., even at 2 000 lb. per sq. in. stress in the column, the deformation becoming so great, in fact, that any vertical reinforcing steel, unless in such quantity as to take the full load, would pass its elastic limit soon after the point of first crack,† and by its buckling increase the surface peeling. Furthermore, from the appearance of the deformation curve, the concrete itself would seem to be in somewhat the same condition as is steel after it has passed its elastic limit.

When it is considered that the usual practice in concrete column design takes no definite account of eccentric loading, or of bending caused by expansion and contraction of floor and wall areas, and that inferior spots may occur in any concrete, through careless mixing or placing, it appears that the greatest care should be exercised in fixing the unit stresses in hooped columns.

Tentative conclusions with regard to hooped column design at the present stage of tests may be summarized as follows:

(1).—Hooping, if properly applied, increases the ultimate breaking strength under a single loading to double or treble the breaking strength of a plain column.

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* *Proceedings*, American Society for Testing Materials, Vol. VII, 1907, p. 382.

† See also Mr. Howard's tests, in "Tests of Metals," U. S. A.

Mr. S. E. Thompson.

(2).—The surface of concrete outside of the hooping will begin to crack at a loading corresponding to the breaking load of an unhooped concrete column.

(3).—Hooping, if not continuous or rigid, will peel off with surface concrete, so that the effective strength of the column will be no greater than a similar one of plain concrete.

(4).—The total vertical deformation of a hooped column is so great at the period of first external crack that any vertical steel, unless designed to carry the entire load, is stressed beyond its safe strength.

(5).—The ultimate breaking strength of a hooped column is no measure of its safe strength, and formulas based on the ultimate strength are useless.

(6).—With the present knowledge, based on tests in America and abroad, the safe load allowed on hooped columns should be but slightly, if any, greater than on similar columns without hooping.

In spite of the favorable reports which have resulted from the European experiments upon hooped concrete, it seems impossible to ignore the additional facts brought out by American tests. Before the hooping acts, the concrete has begun to crush, and any structural material which has begun to crush is unsafe.

Mr. Burr.

William H. Burr, M. Am. Soc. C. E.—Statements made in the course of this discussion appear to indicate that, in such a general treatment of the entire concrete-steel question as this, some features at least of the use of concrete-steel should receive a more careful consideration than would otherwise seem necessary, in view of recent successful constructions.

Caution has been urged against using a unit working stress in the concrete-steel combination exceeding one-tenth of the ultimate resistance of plain concrete, such caution being based upon some of the results obtained in the tests of 12-in. cubes of 1:2:4 concrete at the Watertown Arsenal. In the consideration of experimental results attained by testing concrete cubes, it is of the utmost importance to know completely all the circumstances of such tests, including the preliminary tests of the cement used and the gradations of the sand and gravel or broken stone aggregate. If a 1:2:4 concrete should be mixed relatively dry, and allowed to set in air and remain in a dry building, from the time of its mixture until testing, the results at the end of any usual test period might and probably would be quite different from those found at the end of the same period with a comparatively wet mixture kept constantly moist by sprinkling for a month or longer subsequent to mixing. Other conditions equally productive of varying results can be named, besides the quality of the cement.

As a matter of fact, there are numerous tests of 12-in. cubes of 1:2:4 concrete in the records of the Watertown Arsenal which show an ultimate compressive resistance of from 3 000 to 3 600 lb. per sq. in.,

Mr. Burr. and even more, at the end of three months, with increasing resistances for longer periods. It is a conservative statement to say that well-balanced 1 : 2 : 4 concrete, made with a good quality of Portland cement, may give from 2 700 to 3 000 lb. per sq. in., at the end of three months, with ultimate resistance continually increasing with age. Such concrete may properly and safely be expected to reach ultimate resistances of from 4 000 to 4 500 lb. per sq. in. at the end of a year, results which are justified by extended experience both in America and in Europe.

It is difficult to assign any satisfactory reason for the use of a working stress as low as one-tenth the ultimate resistance of concrete. It is true that there are occasional cases of retrogression, but, with the high grade of Portland cement available from the most reputable producers both in America and abroad, it is reasonable to state that, with the usual engineering inspection to which the best classes of public work are now subjected, cement with retrogressive qualities may confidently be excluded. No engineer at the present time need apprehend sensible difficulty in securing Portland cement the resistance or strength of which will go on increasing indefinitely, and, having reached its maximum, hold it. Under such conditions, a working resistance or permissible intensity of compression in concrete of one-fifth to one-sixth of its ultimate, certainly affords all margin of safety required for engineering works of the best class. Indeed, probably a somewhat higher working stress than that is justified in large structures of reinforced concrete, especially where the reinforcement is of such a character as to give material lateral support to the concrete. This subject is illustrated effectively by the report of a French Government Commission bearing upon the use of reinforced concrete in France. In that report the limit of compressive stresses allowed in reinforced concrete is two-sevenths of the ultimate crushing resistance of the same concrete as determined by tests of plain cubes at the age of 90 days, with the further provision that this two-sevenths may be increased to three-fifths if the longitudinal and transverse reinforcements comply with certain prescribed conditions. This French provision would yield a safe working stress with first-class reinforced concrete work but little if any under 900 lb. per sq. in. The Bureau of Buildings of the Borough of Manhattan, New York City, therefore, has taken a safe and satisfactory course in allowing 750 lb. per sq. in. in such reinforced concrete work as the Thirty-ninth Street Building in the City of New York. In fact, this latter working resistance is conservative for the best class of reinforced concrete work of the present time.

The apprehension regarding the reliability and durability of reinforced concrete work as shown by timorous expressions reminds one strongly of the attitude which some engineers and others used to take toward structural steel when it first came into use, twenty-five or more

years ago. It is remarkable, when one reflects that structural steel is practically the only structural metal which we now possess, that at the period to which allusion is made it was frequently argued out of any future possibility of use, as compared with such a reliable material as wrought iron, in consequence of the erratic behavior which some structural steel members exhibited at that time. Fine cracks, started at a punched rivet hole or sheared edge, would sometimes extend far enough to destroy the reliable carrying power of a channel or angle or other member. Such disclosures, with other erratic experiences, were sources of keen apprehension to many; others, however, believed them to be merely passing phases of difficulty, which attend the introduction of all new materials and processes, and careful study, with intelligent shop manipulations, has shown them to be such. Experience, of course, has more than justified the advocates of structural steel, and that metal has now proved to be, not only reliable, but by far the best structural material ever yet made available to the engineer for a wide range of purposes; indeed, wrought iron is no longer available for structural purposes, nor has it been for a number of years. Mr. Burr.

Reinforced concrete is passing through a similar phase. It is admirably adapted to a great range of structural purposes. Much has already been learned in regard to it, but extending experience will disclose a widening fund of information of value to the engineer in its intelligent application. As a matter of fact, more is actually known about the carrying capacity or the ultimate resistance of concrete-steel members than about the carrying capacity of steel columns, as determined by actual tests. There has already been accumulated a great mass of well-considered and well-digested experimental data regarding the design and construction of both concrete-steel beams and columns, although there is need of many additional tests of some of the latest and best forms of concrete-steel columns. On the other hand, there are almost no tests of full-sized steel-built columns, made in such a way as to disclose some of the most important fundamental principles of design. In the present condition of actual tests of the two classes of members, it is reasonable to believe that there may be at least as much confidence attached to the computed ultimate carrying capacity of both reinforced concrete beams and columns as now built under the best design as can be attached to the computed ultimate carrying capacity of steel columns. Engineers have been so accustomed to design and construct built-steel columns in their every-day work that few ever reflect on the paucity, or even absence, of experimental data on which to base a rational and competent design of such members.

All that reinforced concrete construction needs for reliable results is good cement, good inspection, and intelligent design, which, up to the present time, it has not always had. It is one of the most useful build-

Mr. Burr. ing materials which the engineer has yet had at his command, but it must be dealt with in a manner suitable to any first-class engineering work. There must be rational design, intelligent and effective handling, and good inspection, precisely as with structural steel; and, under such conditions, reliable and durable results may confidently be expected.

Mr. T. K. Thomson. T. Kennard Thomson, M. Am. Soc. C. E. (by letter).—Reinforced concrete, like all other good things, should be protected from its friends. Many young men, having very little knowledge of steel or concrete, have formed companies to build reinforced concrete structures, and one of the first things with which they come in contact is the fact that to obtain a contract they must bid low, another is the necessity of showing the advantages of reinforced concrete over structural steel, and, as the question of cost is the one that appeals most forcibly to the majority of purchasers, they try to design their structure so that the cost will be as low as, or not much higher than, plain steel. One of the methods of doing this is to use fiber strains which are higher than a good bridge or building designer is accustomed to allow.

Many who design reinforced concrete strain their steel bars up to 20 000 or 22 000 lb. per sq. in.—strains which bridge engineers have countenanced only for very long spans, that is, those where the dead loads are large compared with the live load. The recent collapse at Quebec, where it was intended to allow a possible strain of 24 000 lb., and where, owing to faulty detailing, the structure failed at about 18 000 lb., has made many doubt the wisdom of allowing such high combinations of strains (even if only possible), which are hardly likely to occur on any span.

It is practically impossible to ascertain the exact elastic limit of the built-up members of a bridge—due to imperfections of workmanship, material, etc., etc., and therefore it is decidedly unsafe to approach too close to the elastic limit, in estimating the stresses, or to assume that the elastic limit of the test bar is the elastic limit of the full-sized member. There is no reason for allowing higher fiber strains in reinforced concrete than in plain steel, as there are many elements of uncertainty in the former which do not occur in the latter, because far more care is required in the field work and inspection of concrete.

One source of danger, "dry concrete," is rapidly disappearing, for dry concrete practically required an inspector for each laborer, in order to ensure proper ramming, whereas wet concrete will almost ram itself—the only danger being the risk of letting the water escape, thus carrying the cement with it. A 4-in. reinforced concrete wall in New York City was recently removed, when it was found that there was no bond between the steel and the concrete. Not knowing the conditions under which the wall was built, it can only be assumed that the concrete must have been put in too dry.

After the design for a reinforced concrete structure has been made, the three most important considerations are the proper handling of the material, protection from rust, and—more important still—protection from electrolysis. Mr. T. K. Thomson.

In ordinary structures where large masses of concrete are used, buckets containing 2 cu. yd. can be dumped in place, and, if wet, require almost no handling, but, in most reinforced concrete structures comparatively little material is used, and the utmost care is required in handling. In many cases the extra sum paid for labor plus the reinforcement makes the work cost as much as, or more than, a good plain concrete structure containing twice as much concrete, in which case it is better to put one's money into the material rather than into the labor.

Much difference of opinion exists as to whether or not concrete can be made water-tight. The writer's experience has been that it can be, but may not always be, owing to carelessness, and that the mixture should always be rich, that is 1 part of cement to 2 parts of sand, with as much stone as can be covered.

The writer has seen 24-in. I-beams, which had been buried in concrete under the city streets for five or six years, taken out cleaner than they were put in, and in many places showing the original blue shop scale—no paint or oil having been used. In a few isolated places, however, these beams were pitted with rust, showing where the water had found its way to them. It is well known that paint and oil interfere with the bond between steel and concrete. Steel caissons and coffer-dams have been sunk in quicksand in New York City, which, when exposed some seven years later, showed not the slightest evidence of rust.

The writer has removed old steel and cast-iron columns, which had been bedded in concrete and brickwork for years, which showed absolutely no sign of rust. Therefore, in large buildings, carefully constructed, it would seem that there is almost no danger of rust, but it is doubtful if this is true of reinforced concrete bridges, where thin layers of concrete are used, for it has been found very difficult to put in a roadway floor which will not allow any water to percolate through.

The danger from electrolysis is probably very much greater than from rust, and its action is more rapid. There have been cases in New York City where a certain amount of current has been grounded through the steel in foundations buried in concrete, and the steel has been absolutely destroyed. For foundations, it would seem to be safer, in many cases, to rely on mass concrete rather than on thin slabs of reinforced concrete, which cost almost as much in the first place. Of course, in cases where water can reach the embedded steel and carry an electric current with it, the danger is very great.

Mr. T. K. Thomson.

It is probably true that steel in reinforced concrete is much less likely to rust than in a steel structure covered with the best paint, but the latter can be inspected and the former cannot.

In short, the best friends of reinforced concrete should restrict its use to its legitimate spheres, which are many.

Mr. Krellwitz.

D. W. KRELLWITZ, JUN. AM. SOC. C. E. (by letter).—Probably the most novel form in which reinforced concrete has been used is in transmission-line structures.

One case is the 12-mile transmission, for many thousand horsepower at high voltage, from Decew Falls to Welland, Ont., Canada, for which a line with reinforced concrete towers was completed in 1907. Another example is the line of towers* carrying transmission circuits of high voltage to St. Catharines, Ont. These towers are at present the highest monoliths that have ever been erected, being considerably more than twice the height of any of the famous Cleopatra needles.

For the elevations above ground at which it is common to support the conductors of transmission lines (from 25 to 45 ft.), a reinforced concrete tower, in various parts of the United States and Canada, will cost from one to five times as much as a wooden pole. It follows at once from this fact that there must be cogent reasons, apart from the matter of first cost, if the substitution of reinforced concrete towers for wooden poles on transmission lines is to be justified on economical grounds. The electric transmission of energy from distant waterpowers to important centers of population has grown from the most humble beginnings to the delivery of hundreds of thousands of horsepower in the service of millions of people, and the lines for some of this work are supported on reinforced concrete towers. Electrical supply in Buffalo, N. Y., to the amount of 30 000 h.p., depends entirely on the circuits from Niagara Falls which operate at 22 000 volts and, at Tonawanda, N. Y., are supported on reinforced concrete.

In the operation of high-voltage transmissions, during the past, some difficulties have been met, but they have not been so serious as to prevent satisfactory service. Nevertheless, it is being urged that certain impediments, met in the operation of transmission systems, would be much reduced by the substitution of reinforced concrete for wooden poles, and it is even suggested that perhaps the first cost, and probably the last cost, of a transmission line of this kind would be less than with wood for supports. The argument for reinforced concrete, in the matter of costs, is that, while a tower requires a larger investment than a wooden pole, yet the smaller number of towers may reduce the entire outlay to about the same as for wood. More than this, it is said that the lower depreciation and maintenance charges on rein-

* Described by the writer in his paper on "Reinforced Concrete Towers," *Transactions*, Am. Soc. C. E., Vol. LX, p. 160.

forced concrete supports will make their final cost less than that of wooden poles. Mr. Krellwitz.

One advantage of reinforced concrete over wood is that it will not burn, and is probably not subject to destruction by lightning. The fact that reinforced concrete will not burn may make it desirable in places where a long line passes over a territory covered with brush or timber. In tropical countries where insects rapidly destroy wood the use of reinforced concrete, even at a much greater cost, might be highly desirable.

Mr. Waite. GUY B. WAITE, M. AM. SOC. C. E. (by letter).—Reinforced concrete has its uses, and, up to the present, there are few things to which it has not been found to apply.

Public opinion has changed within a very few years from serious doubt about concrete being good for anything to that now held, that it is good for everything.

Friends of concrete can do much damage to the cause by insisting on pointing out personal achievements where actually failures should have been recorded.

It is not possible for one man to formulate a statement as to the universal adaptability of concrete for a given purpose, in all localities from New York to California, without a knowledge of all the conditions in each locality. The popular idea seems to be in most places that concrete should be used for buildings because it is so much cheaper than wood, and that in concrete construction the cost of almost anything is very trifling. This view has recently been strengthened by one of our most distinguished and respected prophets, who promises to see that a two-family house, if it is desired, is turned out complete in a few hours. It is to be regretted that the necessary details to enable others to benefit by his discovery are not disclosed.

Concrete has its *pros* and *cons* which could be stretched in long columns, thus, for example:

Against concrete:	For concrete:
Not good in tension;	Good in compression;
Requires forms;	Good for limited amount of shear;
Requires time to set;	Strength improves with age;
Difficult to tear down—or to fall down;	Economical where forms are simple;
Etc.	Is monolithic;
	Etc.

Stone concrete, mixed in the proportion of 1 : 2 : 4, can be laid down in almost any part of a fair-sized building, with profit, at 30 cents per cu. ft., not including forms.

An average steel column, for a corresponding building, could be erected, at a profit, for $90 per ton.

Mr. Waite. Average steel floor beams and girders, of standard sections, will cost $60 per ton.

Beginning with the supporting columns of a building, a properly reinforced concrete column (conservatively estimated) will carry an average of 750 lb. per sq. in. On the other hand, suppose the corresponding steel column to be strengthened so that it carries an average of 16 000 lb. per sq. in. Then the required amount of materials in the two cases will be as 750 to 16 000, or about as 1 to 21.

The costs of corresponding sections of the two materials, on the foregoing assumption, will be 30 cents and $21.96, or as 1 to 73. Therefore the relative costs of the sections of each material to carry any unit loading will be as 21 to 73, or about 1 to 3½ in favor of the concrete column.

From here on, practical experiences will become useful to decide whether the percentage of 1 to 3½ in favor of concrete is the ultimate ratio of cost, when everything is considered.

Even engineers prejudiced in favor of steel will perhaps concede that for this steel column about 12 to 15% will have to be added to the carrying shaft for fittings, etc. (and in the case of latticed columns much more than this), which added amount of steel will be sufficient to reinforce the concrete column—according to the accepted theory of hooping. Further, if the steel column is to be protected from rust as well as fire, the forms and the concrete material for such fire-proofing will be substantially the same in each case.

Without taking time to go further into details, it would appear that concrete properly used in the form of columns would certainly have the better of the argument, when comparing costs.

The speed of erection sometimes becomes important, and, where the reinforcement to the concrete column is made in the form of an independent carrier, construction can proceed approximately as rapidly as in all-steel construction.

The next objection to the concrete column is naturally the increased size. This objection cannot be raised consistently except in normal buildings more than six stories high, and this in the lower stories only. If the buildings be eight stories high, the size of the columns will only be abnormally large in the two lower stories, etc. A well-constructed building, six stories high, should have columns of steel of not less than a certain outward dimension, in order to give proper rigidity to provide against eccentric loading, etc., and such steel columns, when fire-proof, will be substantially of the size of the solid reinforced concrete column, with an equivalent strength and rigidity.

With development along the lines of improved reinforcement for the concrete, in reinforced concrete columns, it is believed that in the future the sizes of concrete columns can be reduced to meet all requirements.

Concrete, in connection with reinforced floors, is usually taken with a working stress of 500 lb. extreme fiber strain. Mr. Waite.

With the usual T-section of floor construction, an average working load on the entire sectional area for compression can be taken safely at 450 lb. per sq. ft. Estimating the steel beams to take this load at the unit prices set forth above, the comparative costs of concrete and steel would be about as 35 to 49, showing an economy in favor of concrete, other things being the same. But, in this item of floor construction, the concrete floors have to be installed, even when the all-steel construction is used, in order to coat the steelwork and protect it against rust as well as fire. So that, in reality, if the concrete cost as much or more than the steel doing the compression work, whatever is saved by putting this concrete to work is a clear gain, other things being the same.

Other things do not always remain the same, however, and it is necessary to consider the form work for the reinforced concrete construction and the forms for the fire-proofing, used when steel construction carries all the floor loads.

With the steel beams and girders giving the working lines and offering ample supports for the wood forms, the modern system of forms for fire-proofing is very materially less than where much stronger independent framings and supports must be carefully leveled and supported for the reception of the concrete, in reinforced work.

Where the forms can be made in the same general manner as fire-proofing (as in some improved systems of reinforced concrete), the discussion of the relative costs of forms can be dropped, and one may proceed to compare other items in the relative costs of concrete and steel constructions. Now, assuming that forms are the same, and that the concrete is used as a fire-proofing in each case, showing a gain for every bit of the concrete in the reinforced scheme (which is not obtained in the fire-proof scheme), then, if it is not clear that there is economy in the reinforced scheme, it is because the concrete can be made cheaper in the one construction than in the other. The floor slabs will have the same loads to carry when acting as carriers from beam to beam: the concrete, to be an effective protection to the steel against deterioration, must be rich, so that, if the ultimate objects are to be accomplished, the concrete should be substantially the same in either case.

Without making the inquiry more monotonous, it would appear that, in floors, concrete reinforced construction shows an economy in proportion to the amount of steel it is able to replace. So that, where economy alone is the object, a good steel job is necessary. In light constructions (such as dwellings and hotels), where but little steel is necessary, one cannot save as much by using concrete as where the steel is heavier; and the saving continues to increase with the amount of steel to be saved.

Mr. Waite. In the foregoing comparison of relative costs in column and floor constructions the form work is similar whether reinforced concrete or steel and fire-proofing be used. In monolithic wall and partition construction the comparison is disadvantageous when it is considered that brick walls and partitions are laid rapidly and without the inconveniences of forms, and that double forms are necessary for concrete. Further, it is very much more difficult to place the forms for straight walls or partitions than for either columns or floors. The wall forms are not easily held plumb, or in straight lines. The removal of the forms for walls is also much more difficult than for either columns or floors. The cost of common brick and mortar amounts to about 18 cents per cu. ft., and the cost of the materials composing concrete is just about the same. So that the cost of laying the brickwork, for walls of the same thickness, must be balanced by the cost of the double forms and placing the concrete.

It is not intended to burden the reader with descriptions of the difficulties of constructing form work for vertical structures; but, to anyone having much experience, it must be evident that such difficulties must be met. Economy in wall work must be looked for only in heavy work, where the quantity of material placed for any given form is sufficient to pay for it, without materially affecting the cost of the concrete.

When no finish is looked for on the concrete work, rough forms may be placed for from 4 to 5 cents per sq. ft. on each side of the wall; but, for good form work, the cost will run from 7 to 10 cents per sq. ft. on each side.

Concrete walls will be erected. They are an improved construction, and can be handled conveniently in connection with other concrete work in a building. The object of writing what seems to the writer to be the truth about their construction is that economy in their construction should be looked for along other lines than making double forms for the reception of the concrete. It is believed that there will soon be other means of erecting concrete walls and partitions, in which concrete can more than compete with the rapid and economical brick wall.

Mr. Slocum. C. L. Slocum, Assoc. M. Am. Soc. C. E. (by letter).—The science and use of reinforced concrete in the United States appears to be in its earlier stages, as compared with a longer and more thorough acquaintance and varied use in Europe. Only recently its wide application in America has been appreciated in the manifold kinds of construction which are now seen almost everywhere. Generally speaking, theory and practice do not seem to be as closely allied in America as abroad. American engineers have not learned, as well as European engineers, that knowledge of the constituent materials and thoroughness in details of construction are more important than records in speed of erec-

Mr. Slocum.

tion. Like everything new, much opposition, in the nature of incredulity, has to be overcome. For its age, reinforced concrete is fairly well understood, and it may be said that its newness is its greatest fault. The change in the field of design caused by the knowledge of the properties and capabilities of the combination of concrete and steel is now general, and is somewhat in the nature of a revolution in construction. There is hardly a department or particular sphere of construction which has not been changed by it. Homely and incongruous constructions in wood, steel, and stone, and other types of construction too highly commercialized, may now, at reasonable cost, give place to permanent structures, which are pleasing to the eye and are harmonious additions to the locality or landscape. Many types of construction in vogue or considered as good standard practice two or three years ago are now appropriately known or should be known as a part of the history of construction.

If reinforced concrete can be accorded the same conscientious treatment and scrutiny as steel receives, there need be no hesitation about making the change to more permanent and artistic structures, which, if honestly built, will cause no concern or attention after they are put in place. The mature design and construction of steelwork to-day is accomplished by experts in that line, and these are necessary accompaniments of its age and maturity. The use of reinforced concrete needs more rigid inspection in construction, for it is idle to apply carefully intricate formulas to designs which when constructed suffer for want of expert superintendence and experienced labor.

In the realm of bridge construction, where ample depth is available, there is not much doubt as to its economy. This still holds true for spans with comparatively shallow depth, and with light loads, in the nature of moving concentrations. For crossings with little depth of structure available, with heavy moving concentrations, its sphere of usefulness is at present advisedly confined to short spans. However, even floor spans, up to and from 30 to 40 ft., under heavy concentrations, with less than the ordinary depth, can well be investigated. Fabricated units, of simple shapes, as reinforcement, with little or no shopwork, will afford ample stiffness. Theoretical analysis, however, must show that the unit stresses in the concrete and steel are well within the fatigue limits. Continuous framework or an interdependent system of units, easily put together, as reinforcements, but rigid in itself when complete, would seem to afford as much stiffness as steel beams bedded in concrete, which are generally calculated as carrying all the loads independently. In true reinforced work the homogeneous combination of the concrete and steel is the supporting resistance. The full use of the two materials to carry the loads must be more economical than the use of the one which has the concrete merely as a protection. The writer doubts the economy of hybrid construction.

Mr. Slocum. With old or much-used material the internal or molecular structure and properties of which have been changed, or are in doubt, half the usual unit stress allowed for new reinforcement, or doubling the usual economical percentage of reinforcement, would seem to be safe and advisable. The use of old material, of cumbersome, as well as dubious section, of, say, 4 or 5 sq. in. net section, such as old rails, is inadvisable for floor bridges in total length greater than the commercial rail lengths; because attempts to develop such sections in tension are too expensive, and are somewhat abortive.

In a series of short, independent, self-supporting arches of reinforced concrete, which are very flat, and are practically carried on continuous columns, the writer has used the cantilever method in finding the stresses in the constituent materials, and has proportioned the steel accordingly; in other words, he has considered the middle third of each span as carried by the end thirds. These arches were calculated for the heaviest moving concentrations for highways. In beam and slab bridges, carrying heavy trolley concentrations, where the design is somewhat hampered for depth, higher percentages of steel and double reinforcement may have economical advantage.

In current American practice, more time can be allowed to good advantage for this construction to attain mature strength rather than use a green structure prematurely and perhaps lessen the efficiency of the bond. Collections of materials of construction or equipment, sometimes inadvertently placed on new work, give concentrations for which the design is not calculated, and, if the work is not of sufficient age, much damage may be done, and may not be evident until some time after. Such consequent weakness may be brought out by fatigue, which, under normal conditions, could not be explained. From observation, competent, well-paid superintendence and experienced workmen of the best class give the strongest structure and the one that fulfills all the conditions of economy.

As compared with the usual heavy masonry arches of gravity section, the comparatively light reinforced arches give more and greater vibrations under moving loads, principally on account of much less bulk weight of structure. Can reinforced concrete work vibrate with the same impunity as steelwork? The writer thinks it can, if the working stresses are not too high, but are well within the fatigue limits. Much interesting and instructive information could be obtained by measuring the number of vibrations and their amplitude on bridges of different types under different kinds and speeds of rolling loads. Under any conditions, crossings of shallow floor construction can well be tested for unusual loads, and consequent deflection, if any.

Other properties and characteristics being satisfactory, a greater proportion of finely-ground cement, with a graded aggregate will, with safety, give reinforced concrete design and construction its bold quality,

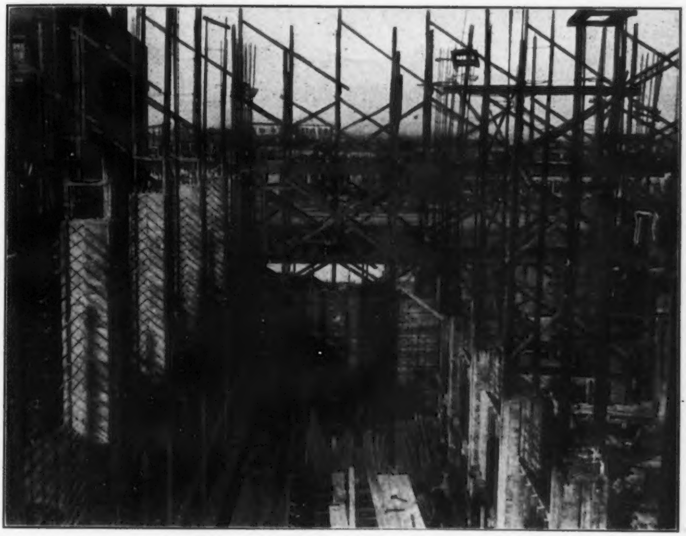


FIG. 1.—REINFORCED CONCRETE STRUCTURE FOR ICE STORAGE.

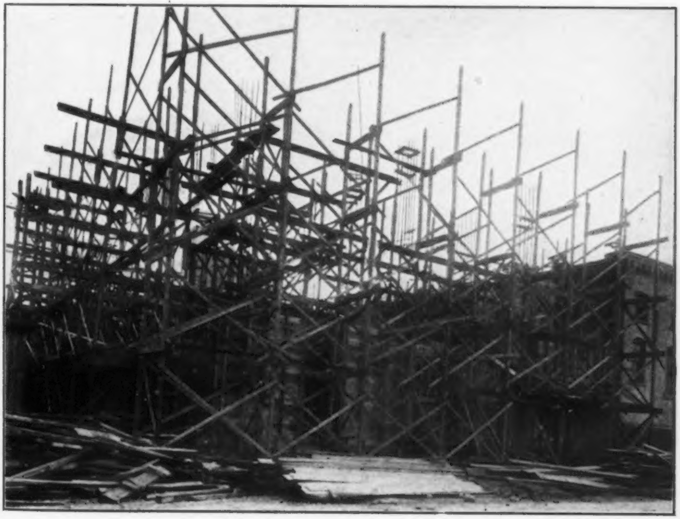


FIG. 2.—SCAFFOLDING TO SUPPORT REINFORCING RODS.

which distinguishes it. Too little attention is paid to the compactness or density of the mixture. The result of a few simple and inexpensive experiments in the measurement of voids, taking a comparatively short time to perform, will give a cheaper and stronger concrete. In reinforced work, such preliminary investigations are productive of economy. Mr. Slocum.

Myron S. Falk, Assoc. M. Am. Soc. C. E.—A considerable number of reinforced concrete structures have of late been described with enthusiasm before this Society and in the technical press; and many, if not all, of the published descriptions make it appear that these structures have been a complete success from the time of their inception, causing no trouble to designer, owner, or contractor. Mr. Falk.

As a rule, these descriptions cover the completed structure only, and omit references to the difficulties and dangers encountered during construction.

Candid statements of facts in relation to the use of reinforced concrete are absolutely necessary at the present time; such statements must be accurate, and should conceal nothing, so that they may serve as guides to others who propose this construction for similar classes of work.

Plates IV and V illustrate the construction of two buildings, entirely of concrete, which were built during 1907, are now in use, and, to any observer, would appear to be eminently satisfactory. In neither case will the respective owners repeat their experiences, since in both instances they have learned that different methods would have afforded structures which could have been erected more quickly, at less cost, and would have been fully as permanent.

Plate IV and Fig. 1, Plate V, show an ice storage house, the outside dimensions of which are 58 by 92 ft., and with a clear inside height of 42 ft. 8 in. from the top of the basement floor to the underside of the roof slab. The columns supporting the roof are 18 by 12 in. in cross-section, and are embedded at intervals of about 11 ft. in the curtain walls, which are 12 in. thick for the exterior and 10 in. for the interior walls. The building, which is to store cakes of manufactured ice, contains three chambers running the full length of the structure, the only entrance to each being through a small ice chute in the front of the building. There are no windows. At first study, any engineer would claim such a structure to be ideal for reinforced concrete; forms for vertical walls and for one roof slab only were required. The history of the case, however, refutes this.

The building was planned by an architect, who called for proposals, requiring the bidding contractors to design the reinforced concrete work subject to his approval, although he himself wrote the specifications under which the work was to be built, and prepared preliminary plans showing his ideas as to reinforcement. One of the requirements

Mr. Falk. was that the walls and columns were to be designed to withstand an assumed horizontal thrust due to the pressure of the ice. Consequently, the columns were designed, by the contractors to whom the work was awarded, as vertical beams loaded at their centers with the ice thrust. This explains (Fig. 1, Plate IV) why the reinforcing rods in the columns were placed in two lines parallel to the exterior faces of the columns, instead of being spaced more uniformly throughout the cross-section. The rods forming this reinforcement ran, for the most part, the full height of the building, and, as they were not self-supporting, it was necessary to build a wooden structure to hold them before any concreting work was done. This structure is shown in Fig. 2, Plate IV.

The rods in the columns were hooped together at short intervals, not only by outside wires, but also by wires crossing through the center; moreover, in order to space the corner column rods away from the forms, the contractor inserted plate-washers on each corner rod.

The curtain walls were also reinforced with horizontal and vertical rods spaced and wired as shown in Fig. 1, Plate V. It is evident that the reinforcement acted as a screen through which the raw concrete was forced to pass.

One clause of the architect's specifications read as follows:

"The centering for columns shall not be over half the height of the building before concreting is commenced and for enclosing walls not over 10 ft. in height, unless otherwise approved."

Although the contractor should have known better, he blindly attempted to follow this clause. The final results of the work, taken in connection with the design, are shown clearly in the photographs, and require no explanation, except that, when the forms for the lower portions of the walls were stripped, the owner, mistrusting both contractor and architect as to the safety of the work, called for engineering advice.

The structure was completed, after much difficulty, strictly according to plan; dangerous defects were repaired so that no failure may be expected, and surface blemishes were plastered so that anyone not familiar with the actual construction might believe the building to be an example to be followed.

The building shown by Fig. 2, Plate V, was originally designed by an architect as a frame building to be finished in cement stucco; but a reinforced concrete contractor convinced the owner that it would cost but little more to make the building entirely of concrete, and he was given the order to proceed. In fairness to the architect, it should be stated that he was not consulted as to the building after the original plans had left his hands.

When the structure had reached about half way to the second story the owner began to suspect the character of the work which was being done, and decided to complete the building by day's work in

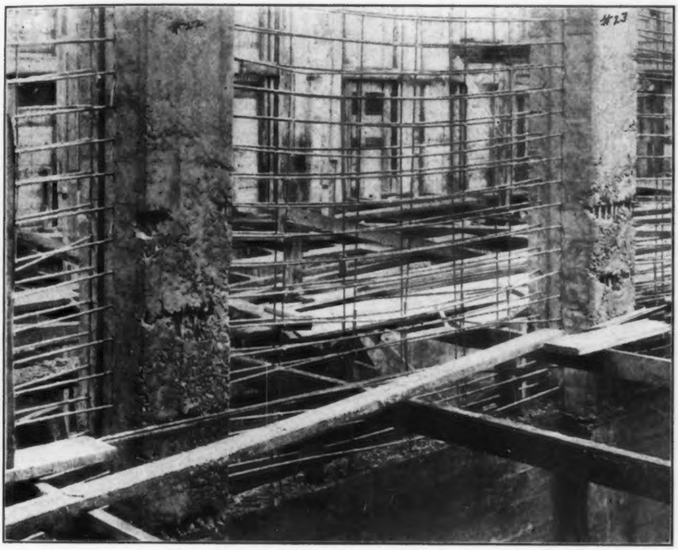


FIG. 1.—REINFORCEMENT OF WALLS AND COLUMNS.

FIG. 2.—REINFORCED CONCRETE BUILDING.

charge of an engineer. No difficulties out of the ordinary were encountered until the roof was reached. Mr. Falk.

The building is 40 ft. square, and there are four interior columns. The roof is a concrete slab, sloping at about 45° from the horizontal, and is supported on the side-walls and on two concrete beams running the length of the building and carried by the concrete columns. The concrete in the main portion of the building had been poured very wet; but when this mixture was placed on the sloping roof forms it refused to stay in place. Therefore, wooden planks had to be placed on top of this concrete in order to hold it down. This method, however, was exceedingly difficult, as the roof was a dangerous place for the workmen. The concrete was changed to a drier mixture, but still required the use of the outside forms. As it was impossible to lay very much of the roof in one day, there were many joints. After the concrete had set so that workmen could move about without injury to it, a surface coat of mortar, in which was incorporated a so-called waterproof compound, was placed. This coat was colored with red oxide of iron, so that the final surface showed a pleasing red. The surface coat was plastered smooth, and it seemed as though all water would be easily shed. The first rain storm, however, showed that the roof leaked almost like a sieve. It must be remembered that this work had been done by day's labor, and not by contract, and that there had been absolutely no incentive for any but the best workmanship.

The speaker consulted several water-proofing companies, asking them to water-proof the roof without destroying the color effect which had been obtained, but not one of these companies would take the work and guarantee it for more than one year. The use of pitch or similar water-proofing material was not permitted on account of the color, nor does the speaker believe that any plastic material would stay on this roof. It was finally decided to apply alum and soap, as in the Sylvester process, and from its application up to the present time the roof has shed the rain. It has not yet passed through both a summer and a winter, and it will be interesting to note what effect the temperature will have on a thin slab of this kind. The speaker would not advise anyone to use a reinforced concrete roof of this kind.

Rudolph P. Miller, M. Am. Soc. C. E.—In the speaker's experience, along the line of building construction, the success of reinforced concrete in engineering work is greatly dependent on thorough and intelligent inspection. Many a good design has been completely defeated because of the lack of proper superintendence. The materials being used at the present day in this kind of work are generally reliable, but their improper handling has often been responsible for poor results. It is desired to call attention here to two defects that have been of too frequent occurrence, which can be avoided by a little foresight in the design and by intelligent supervision in the construction: First, Mr. Miller.

Mr. Miller. the displacement of the reinforcement when the concrete is placed; and second, the formation of cavities in the concrete construction due to complicated reinforcement.

It would seem unnecessary to call the attention of engineers to the danger of the displacement of reinforcing rods or bars in reinforced concrete beams. Concisely stated, if the displacement is upward, there is a loss of strength proportionate to the amount of displacement; if the displacement is downward, the fire-resisting qualities of the construction are impaired, and ability to resist fire is one of the main claims of superiority of reinforced concrete construction. Judging from experience, however, it seems to be important to call the attention of engineers to the necessity of making provision for preventing the displacement of the reinforcement. It is the speaker's opinion that, no matter how carefully bars or rods are placed in the moulds, or what precaution is taken in the pouring of the concrete, there can be no assurance that the reinforcement is in its proper position when the work is completed, unless some means have been used to prevent a movement. The only certain method that has come to the speaker's attention is that used in the so-called "Unit" systems, in which all the reinforcing bars or rods in a beam (and it is equally applicable to column construction), including the stirrups, are secured by heavy wire clamps or other devices in such a way that their relative positions cannot alter. By using washers or spacers the resultant frame can be secured in the forms against a bodily displacement, and held at a proper distance from the outer surface of the finished concrete.

Besides assuring the correct position of the reinforcement, the use of the unit frames greatly simplifies the superintendence of the construction. It requires but a glance (comparatively speaking) to see whether all the reinforcement is in place in the form and whether the proper frame is in each form. The frame having been built up from detailed drawings, previously prepared, the danger of the omission, occasionally, of a bar or rod, of the substitution of a bar of less cross-section, or of the use of too short a bar, is practically eliminated. (See Fig. 1, Plate VI.)

The frames themselves may be fabricated at the shops and shipped to the job; or, if the operation is sufficiently large to justify it, there may be a temporary shop on the premises. The particular advantage in this is that the forms can be inspected and checked before they are put in place. A sample detailed drawing from which the frames are made is shown in Fig. 1.

Fig. 2, Plate VI, shows another and quite satisfactory method of securing the reinforcement in position when the style of columns used is such as to admit of it. This is a photograph of one of the column-girder connections in the McGraw Building, New York, recently described* by William H. Burr, M. Am. Soc. C. E.

* *Transactions*, Am. Soc. C. E., Vol. LX, p. 443.

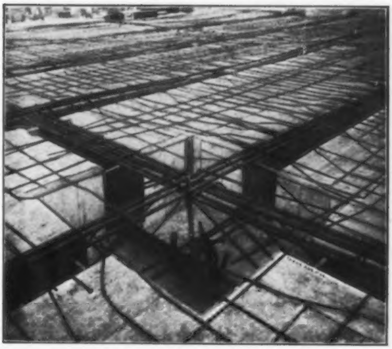


FIG. 1.—UNIT SYSTEM OF FRAMES.

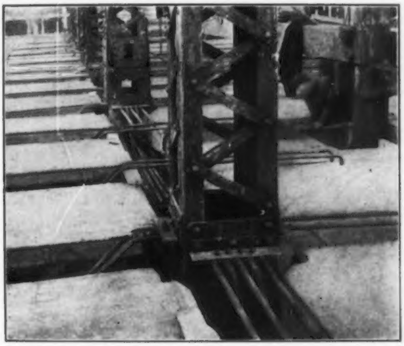
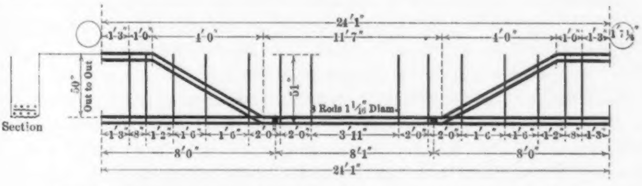


FIG. 2.—METHOD OF SECURING REINFORCEMENT.

FIG. 3.—CAVITIES IN COLUMN.

The second detail of construction which seems to have escaped at- Mr. Miller.
tention is the avoidance of too complicated a reinforcement. In the disposition of the steel, care must be taken that the several elements are not so closely spaced or so arranged as to prevent the concrete from pouring between and around them and thus producing cavities. The size of stone used in the aggregate should be considered in connection with the spacing of the rods, or *vice versa.* When complicated reinforcement cannot be avoided, the size of the stones should be reduced to suit the condition, or the stone should be eliminated, and mortar should be used. All this applies particularly to column construction and other work where the concrete must fall through considerable height. The speaker has seen a column, the cross-section of which was not more than 20% of its embedded area, because of the cavities formed by the sieve-like action of the reinforcement. An instance of what is meant, though not as serious as the case referred to, is shown in Fig. 3, Plate VI.



No. of Pieces	MATERIAL	LENGTH	WEIGHT	REMARKS
4	Rods 1 1/16 Diam.	24' 1"		
4	" " "	27' 4"		
	Single Sockets			
2	Double Sockets			
4	Stirrups 1" x 1/8"	9' 2"		Holes punched 1" from each end
6	" "	8' 10"		
4	" "	8' 9"		
4	Ties 1" x 1/8"	12"		
16	Clamps with Bolts			See Standard Sheet No. Q.

FIG. 1.

EUGENE W. STERN, M. AM. SOC. C. E.—No structural material in Mr. Stern.
recent years has temporarily won such enthusiastic partisanship, or caught the public eye to such an extent, as reinforced concrete. It may be that the reason for this is that it appeals so much to the imagination of the layman.

It is useless to consider all the claims that have been made for it; but one in particular should be flatly contradicted, which is that but little special knowledge is required in the art of working in this material, and therefore that it can be largely done by unskilled labor. In view of the many fatal accidents which have resulted from the improper use of this material, this claim is not as strongly urged as it once was.

A matter of interest in connection with the construction of rein-

Mr. Stern. forced concrete work is that contracting firms, or those who exploit patented or deformed bars, are largely responsible for the designs which go into buildings to-day. They submit their own plans, based on the use of these bars or some special method of construction, under some kind of guaranty as to carrying capacity, almost always without any charge for their services. It is the speaker's experience that this method leads to trouble, very often to a lawsuit. The client's interests are supposed to be looked after by the contractor, but, when any trouble happens, his interests, of course, are forgotten.

It is more than ever necessary, in the use of this material, that the structural design and supervision of the work should be placed in the hands of competent professional engineers who represent the owner's interests only.

In no other material of construction is such extreme care necessary, and such intelligent, constant and painstaking supervision required in every part of the process.

Reinforced concrete is a valuable material for use in structures, and has a large field. It has its limitations, however, and often, owing to the enthusiasm of its advocates, has been used where other materials would have answered the purpose better.

As a facing for buildings it has not proven a success, for the reason that it is difficult to obtain a pleasing surface, and ordinarily is more expensive than brick. In the construction of high buildings, the speaker believes that it is not as suitable for columns and girders as steel-frame construction. For low buildings, or factory and mill buildings, occupying large areas in outlying districts, where there is plenty of room to handle the material, it has proved a very desirable substitute for mill-constructed buildings.

Among the faults of reinforced concrete work is its tendency to crack, due to the shrinkage of the concrete. The speaker has had to deal with a number of reinforced concrete buildings, and none of these has been free from cracks in various places. A recent case was interesting: In a building there were two rows of columns longitudinally, dividing it into three bays. In the center, between the columns, there were heavy girders, but, in the outside bays, for structural reasons, there were in places light girders. The heavy girders, in shrinking, drew the columns slightly together and the outside girders cracked in the top flange.

The illustrations shown by Messrs. Falk and Miller are interesting in showing what actually happens in practice. The speaker, however, has seen much larger voids in columns than any of those illustrated. In one case, where deformed bars having prongs were used, there were voids in the columns which practically occupied the whole area of the column. There was no attempt on the part of the contractor to scamp his work, but the interlacing of the prongs formed a screen which held

up the stone and prevented it from becoming well consolidated in the mixture, with the above result. Mr. Stern.

The use of reinforced concrete for railroad bridges does not seem to be a proper application, for the reason that constant vibration would tend to cause cracks ultimately, and separate the reinforcement from the concrete.

Where conditions would be favorable to the rusting of steel, reinforced concrete is not suitable, unless cracking can be prevented, as otherwise the reinforcement will ultimately rust out.

Mathematical investigations have been carried to an extreme degree of refinement in reinforced concrete construction, and designs have been worked out on paper for huge structures that stagger the imagination of any but the most enthusiastic. Only recently, a design for a bridge over Spuyten Duyvil Creek, New York City, has been prepared by its Department of Bridges, involving the construction of a reinforced concrete arch of 703 ft. span.

Reinforced concrete is anything but an academic proposition. It is eminently a practical one. Theorists assume for their computations certain conditions, some of which may be possible, and some of which may not be possible, to obtain in the practical operation of construction. Many things in the practical use of this material have yet to be understood, and these can only be learned by experience.

Has the state of the art, in the use of this comparatively new material, progressed to such an extent as to warrant the conclusion that, to-day, it is a perfectly safe and legitimate proposition to undertake to build structures, which in magnitude and boldness of conception far exceed anything in existence of similar type? Is it not sounder engineering to progress slowly along well-tried-out lines?

There is wide difference of opinion as to what the working stresses ought to be, particularly in compression. The building codes of the various cities in America are not by any means uniform in this respect, the allowable unit stresses in compression varying from 350 lb. upward, and some engineers have recommended as high a stress as 750 lb. per sq. in.

There have been a great many tests on concrete cubes, the data obtained from which are valuable in this discussion. The highest results have been obtained, of course, where the specimens have been kept in moist sand, or submerged under water; but tests made under such ideal conditions, which rarely obtain in practice, should not be used as a basis for deciding what should be the working stress of concrete in compression, unless these conditions approximate those under which the structure itself is built.

Among the many tests made at Watertown Arsenal, the speaker would refer especially to a series of tests on 12-in. cubes, prepared by the authorities at the Arsenal, the results of which are given in their

Mr. Stern. Annual Reports for 1899 to 1904. These blocks were allowed to set in air, and were stored in a dry, cool building throughout the period of the tests, which were made after periods ranging from 3 months to 5 years. The conditions under which the blocks were stored would be almost identical with those to which reinforced concrete work in building construction would be exposed, and these tests, therefore, would give results more in harmony with actual conditions than those in which the blocks were immersed in water or kept moist in sand for a number of months.

The average	of	10	tests,	after	3	months,	was	1 958	lb.	per	sq. in.
"	"	16	"	"	4	"	"	2 244	"	"	" "
"	"	16	"	"	1	year,	"	3 330	"	"	" "
"	"	15	"	"	2	years,	"	2 610	"	"	" "
"	"	15	"	"	3	"	"	2 610	"	"	" "
"	"	10	"	"	4	"	"	2 960	"	"	" "
"	"	2	"	"	5	"	"	2 630	"	"	" "

Disregarding the 3-month and 4-month tests, the average of 58 tests, after 1 to 5 years, was 2 870 lb. per sq. in. These blocks were all made of 1 part Alpha Portland cement, 2 parts sand and 4 parts broken trap rock, varying in size in the different specimens from ½ in. to 2½ in.

It will be noticed that the 2-, 3-, 4-, and 5-year tests show substantial reductions in strength from the 1-year tests, and the records show, also, that there was a considerable loss of weight, varying from ½ lb. to 2 lb. in each block.

There will undoubtedly be differences of opinion as to what fraction of the ultimate strength should be adopted for a safe working stress. To compensate for the great factor of ignorance which exists in the construction of concrete and reinforced concrete work, there should be an ample margin of safety. No matter what care may be taken with the sampling and storing of cement, it is practically impossible, in the process of construction, to prevent, not only some of the material losing its strength, but also to obviate defects in workmanship.

It has been considered for many years that a factor of safety of from 10 to 20 should be used in masonry. The speaker sees no reason, therefore, why a greater load than $\frac{1}{8}$ to $\frac{1}{10}$ of the ultimate strength of laboratory tests on concrete cubes should be used in practice in building construction, which would give between 290 and 360 lb. per sq. in. as a unit stress.

Professor Burr has brought up the question as to whether or not the Watertown tests quoted by the speaker were all made under uniform conditions and with the same brand of cement and other materials. As far as can be learned from the official reports of these tests, Alpha Portland cement was used throughout, and the same quality of sand and stone; moreover, the specimens were stored under the same conditions, throughout the years during which the tests were conducted.

H. C. TURNER, ASSOC. M. AM. SOC. C. E.—During this discussion, a number of questions have been raised which call for an answer by those who are closely identified with the construction of reinforced concrete buildings. Mr. Turner.

In answer to Mr. Stern's question regarding the preservation of the steel reinforcement in concrete structures, the following is the experience of the Turner Construction Company in razing a one-story building, erected for the J. B. King Company, at New Brighton, Staten Island, in 1902, which was taken down during the summer of 1907 to make room for a larger structure: The building had reinforced concrete walls, 9 in. in thickness to grade line and 5 in. in thickness from grade line to roof line, reinforced concrete interior columns, 11 in. square, and reinforced concrete beams, girders and roof slab. The foundation consisted of spruce piling, cut off at mean tide and capped with reinforced concrete. All steel reinforcement was found in perfect preservation except a few ¼-in. hoops in the wall columns, which were within ¼ to ½ in. of the surface. These showed slight corrosion, which would indicate that it is important to secure all steel reinforcement at least ¾ in. from the exterior surface. The steel in the footings, although alternately wet by the tide each day, was in perfect condition. In some cases this steel was within ¾ in. of the surface.

Numerous observations of a similar kind have been made by engineers, and it is now generally recognized that steel reinforcement is permanently preserved in concrete structures.

It seems unfortunate that illustrations of standard reinforced concrete work have not been shown, rather than those of generally defective work, although such illustrations are valuable in indicating the character of design and workmanship to be avoided. It must not be assumed, however, that they are typical of reinforced concrete construction. Much excellent work is being done in New York City, and in all cities in the United States. Eight and ten-story buildings are not unusual, and it is to be noted that these buildings have proven especially adaptable for heavy storage or heavy manufacturing.

As Mr. Miller has stated, it is perhaps more difficult to secure good workmanship than good engineering design. This is a matter of organization. Good workmanship should be required, and undoubtedly can be furnished. There is abundant evidence of this, and good workmanship costs but little more than poor workmanship. It is necessary to have a thorough and experienced organization of workmen; but this is just as true in any line of successful business.

Regarding safe unit stresses, there is no reason for a factor of safety of ten. Reinforced concrete buildings have a larger factor of safety than steel buildings, because of the monolithic character of the construction. Concentrated loads are distributed over larger areas because the reinforcement extends in both directions. Vibration is

Mr. Turner. largely reduced. This is well demonstrated in the Ketterlinus Buildings, in Philadelphia. The two buildings are about the same size, 8 stories in height; one has a steel frame with hollow tile floors and brick walls; the other, and later, building has reinforced concrete columns, beams, girders and floors, with brick veneer walls. Both buildings are used for printing and lithographing, and are subjected to practically the same floor and machinery loads. The vibration in the concrete building is very noticeably less than in the steel-frame building, in fact, it can hardly be detected.

In the Robert Gair Company Building, in Brooklyn, there is a 16-ton embossing machine set on a 3 by 6-ft. base in the middle of a bay on the seventh floor. No deflection has occurred in the beams, and, when the machine is in operation, no vibration is perceptible, although the working loads assumed for this building were only 200 lb. per sq. ft.

Answering Mr. Miller's observations on the value of unit systems in reinforced concrete construction, the chief objection to them at present is the additional cost, which must be paid by the owners. Unit frames may relieve the architect or engineer of some anxiety and responsibility, but it is admitted that most of the important work in the United States has been done with the loose-bar system; and, with a proper organization, loose bars, so-called, can be placed and secured in the work with absolute reliability. The owner looks for results, and should certainly be entitled to the difference in cost between buildings constructed with loose-bar systems and with unit systems.

Mr. Goodrich. E. P. Goodrich, M. Am. Soc. C. E. (by letter).—Mr. Thacher's collation of the opinions of engineers concerning the effect of sea water on concrete is of value. The trend of opinion seems to be that all good cement when well manipulated will be satisfactory, irrespective of its chemical composition, within ordinary limits. This is in further confirmation of some experiments instituted by the writer, in which briquettes of all the cements delivered were stored in sea water and tested after varying periods. The chemical composition of the cement was known, in each instance, and varied to some extent, but no effect could be traced to differences in this regard.

Mr. S. E. Thompson's deductions with regard to spirally reinforced columns or those supplied with proper bands or horizontal ties cannot be gainsaid, except that such reinforcement absolutely prevents any destructive failure, and that higher working stresses may safely be allowed on such columns than on those containing little or no such reinforcement, in which case dependence must be placed on a wide margin of stress between the permissible and the estimated ultimate, to cover contingencies of workmanship.

Professor Burr's comparison between the present knowledge with

regard to the actual carrying capacities of steel columns and those of reinforced concrete is interesting and reassuring to conservative advocates of the latter. It is of interest to note that the recommendations of the French Commission, quoted by Professor Burr, when applied to cubes 3 months old, as mentioned by Mr. Stern, afford allowable stresses varying between 560 and 1 180 lb., according to the kind of reinforcement used. Mr. Goodrich.

Mr. T. K. Thomson's criticism of the use of high stresses in reinforcement are the more pertinent from the fact that such high stresses are always accompanied by cracking of the encasing concrete, with the involved likelihood of rust and of becoming a source of incipient failure by diagonal tension. The reference to electrolytic action is also of considerable value, and his advocacy of mass concrete foundations is made on strong grounds.

Mr. Krellwitz gives an example of the relative cost of concrete and wood in an exceptional case, but it only goes to show the tendency, and is a prophecy of future uses.

Mr. Waite gives a very fair comparison between the costs of reinforced concrete and steel for floors and columns, and of concrete and brick for walls. It is only through a knowledge of such relations that designs can be economically prepared and a proper selection of materials made.

The compliments accorded American practice in reinforced concrete by many foreign engineers, who have examined it and expressed their views to the writer, as well as his examination of a large number of concrete structures in Europe, lead him to dissent from Mr. Slocum's statements with regard to practice in America and abroad, except that the writer believes that few American engineers pay enough attention to the details of construction or to the inspection of work, in comparison with the amount of time spent on the theoretical design of the main features. Mr. Slocum's remarks concerning the use of old rails, of making simple determinations of voids in aggregates, and of overloading green concrete are particularly apt.

Mr. Falk's candid statement of some of the difficulties encountered in concrete work could very well be brought to the attention of all inexperienced designers. Too many architects and engineers follow blindly when making designs and specifications, and the unsuspecting owner pays the bill.

Mr. Miller's advocacy of unit systems of reinforcement, and of simple reinforcement designs, is sane and timely.

Mr. Stern's protest against the popular idea that concrete work can be executed with entirely unskilled labor is well founded, and that conception should be combatted as often as possible. His further contention, that design and construction should be in the hands of engineers, caring solely for the owner's interests, is also sound. His

Mr. Goodrich. opinion, that working stresses of only $\frac{1}{8}$ or $\frac{1}{10}$ of the ultimate strength of test cubes should be allowed, is believed to be ultra-conservative, in view of the ease with which much higher stresses can be attained with only ordinary precautions. If there is danger of improper workmanship, it is manifestly unjust to saddle the owner with the expense of insurance against trouble by the use of low working stresses. Rather, the contractor should be made to bear the penalty by requiring him to test his work at an early age, at the same time allowing him to use attractive unit stresses if he sees fit.

Mr. Turner's remarks about vibration, and his experience with regard to the amount of cover necessary to protect steel against rust are exceedingly interesting. His advocacy of the loose-bar system would have been more convincing if he had given instances of the comparative cost of the two kinds. The writer believes that almost every owner would be willing to pay 10% more for his reinforcement (or perhaps 2% more for his building) in order to secure the assurance that every bar is in its right place and is of proper size.

The whole subject of reinforced concrete is still so new, and so much missionary work is still necessary to put it on an absolutely sound basis, that all such discussions as this are exceedingly profitable.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1079

THE ELECTRIFICATION OF THE SUBURBAN ZONE OF THE NEW YORK CENTRAL AND HUDSON RIVER RAILROAD IN THE VICINITY OF NEW YORK CITY.*

By William J. Wilgus, M. Am. Soc. C. E.

With Discussion by Messrs. G. R. Henderson, George Gibbs, Arthur M. Waitt, Nelson P. Lewis, H. M. Brinckerhoff, George B. Francis, Edwin B. Katte, W. S. Murray, George A. Harwood, W. B. Potter, Frank J. Sprague, Henry G. Stott, and William J. Wilgus.

The recent successful completion of the electrification of the service of the New York Central and Hudson River Railroad entering the Grand Central Terminal, New York City, marks such an important step in the progress of the art of transportation that a paper seems at this time appropriate, explaining the reasons for the abandonment of steam, the general features of construction and operation, and the results.

Two decades have passed since electricity in the United States first commenced its important career in the field of lighter traffic; but only within the past few months has it fairly met its steam rival in heavy-traction trunk-line service.

Reasons for Delay in Electrification of Trunk Lines.—The reason for this delay is not far to seek. The steam locomotive, during its lifetime of eighty years, has been developed into a wonderfully reliable, efficient, and powerful machine, deep-seated in the affections of

* Presented at the meeting of March 18th, 1908.

the railroad world. With the conservatism naturally born of these conditions is the reluctance of stockholders to spend vast sums for changes of unproven financial value.

There is no cause for surprise, therefore, that electricity, so commonly associated in the mind of the railroad officer with light streetcar traffic, has not been seriously considered as a substitute for steam, until special problems have arisen demanding some escape from the limitations and nuisances incident to the use of steam locomotives.

The very fact that steam locomotives have grown so in size and power makes them more objectionable as emitters of increased volumes of noise, smoke, gas, and cinders.

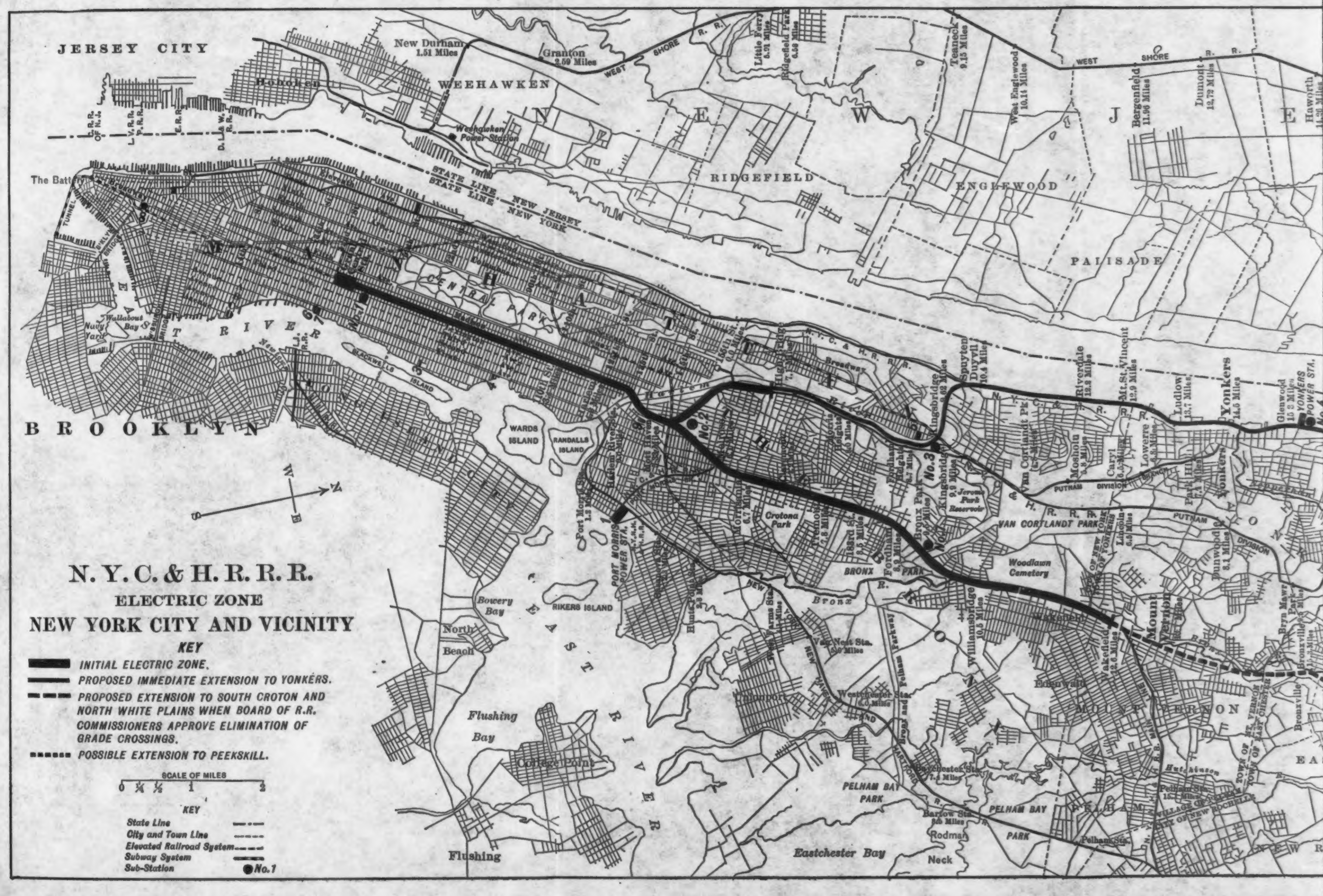
The first important instance of the use of electricity on a large scale was in utilizing electric locomotives to push solid trains, with their inactive steam locomotives, through the Baltimore Tunnel of the Baltimore and Ohio Railroad. In this instance, electricity was adopted as an aid, not as a substitute for steam.

Reasons for Electrification of New York Central.—As early as 1899, thought was given to the use, on the New York Central, of electricity for curing the evils at the entrance to the Grand Central Terminal; but it was not until 1903 that the objectionable atmospheric conditions in the Park Avenue Tunnel, and the congestion of traffic at the terminal, precipitated legislative action directing the complete abandonment of the steam locomotive in Park Avenue south of the Harlem River, within a period of five years terminating July 1st, 1908.

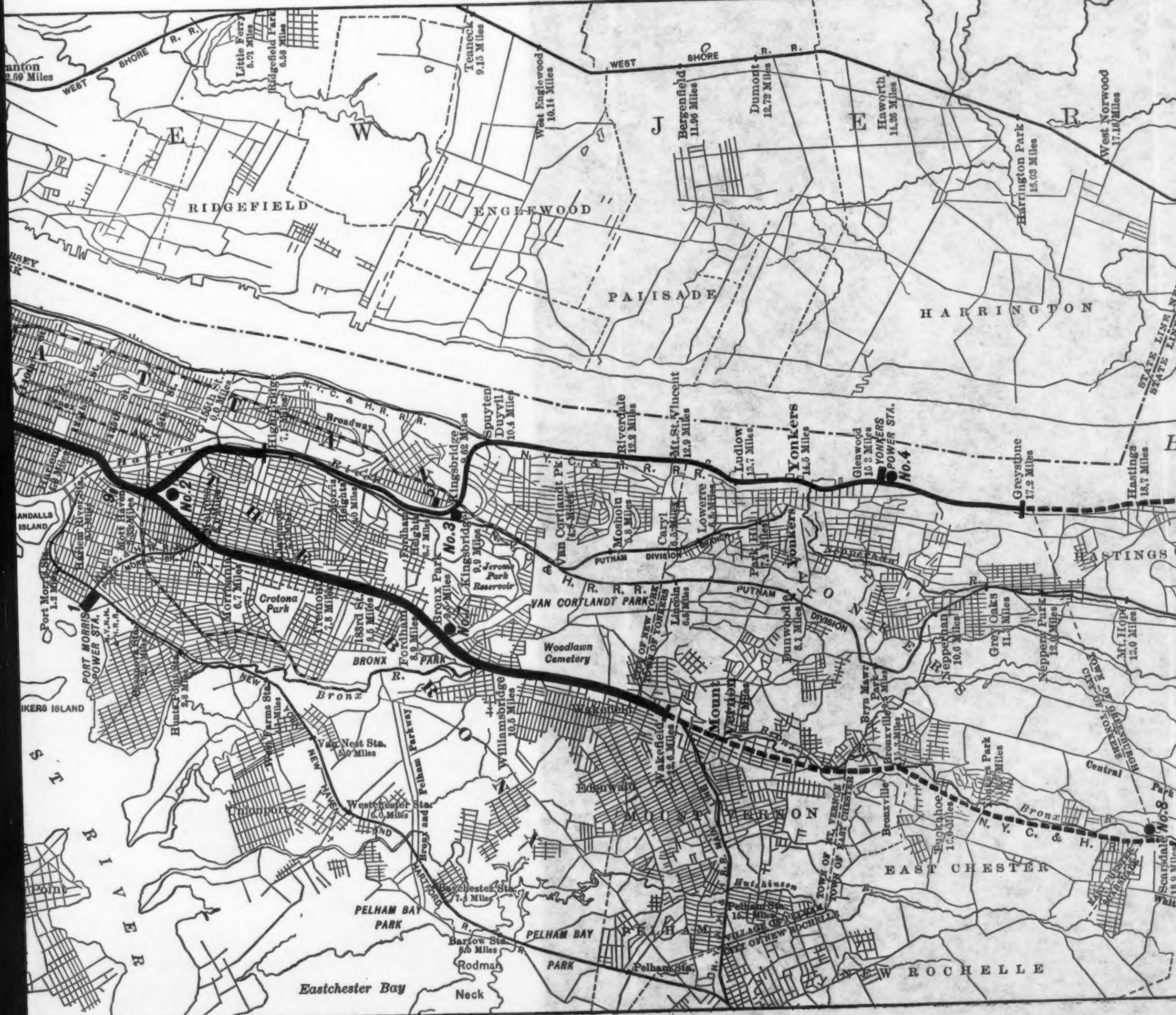
In the same year the railroad company and the city agreed upon radical changes at the terminal, which were possible only with the abandonment of steam. From a civic standpoint, the most important of these changes is the depression of the whole terminal, so as to permit the extension of highways over the tracks from Forty-fifth to Fifty-sixth Streets, inclusive, and the continuation of Park Avenue, 140 ft. wide, within the same limits, thus joining two sections of the city hitherto separated for three-quarters of a mile by an impassable barrier of railroad yards and structures.

Reasons for Extended Scope of Electrification.—A careful analysis of the situation soon proved the absurdity of terminating the electric zone at or near the Harlem River.

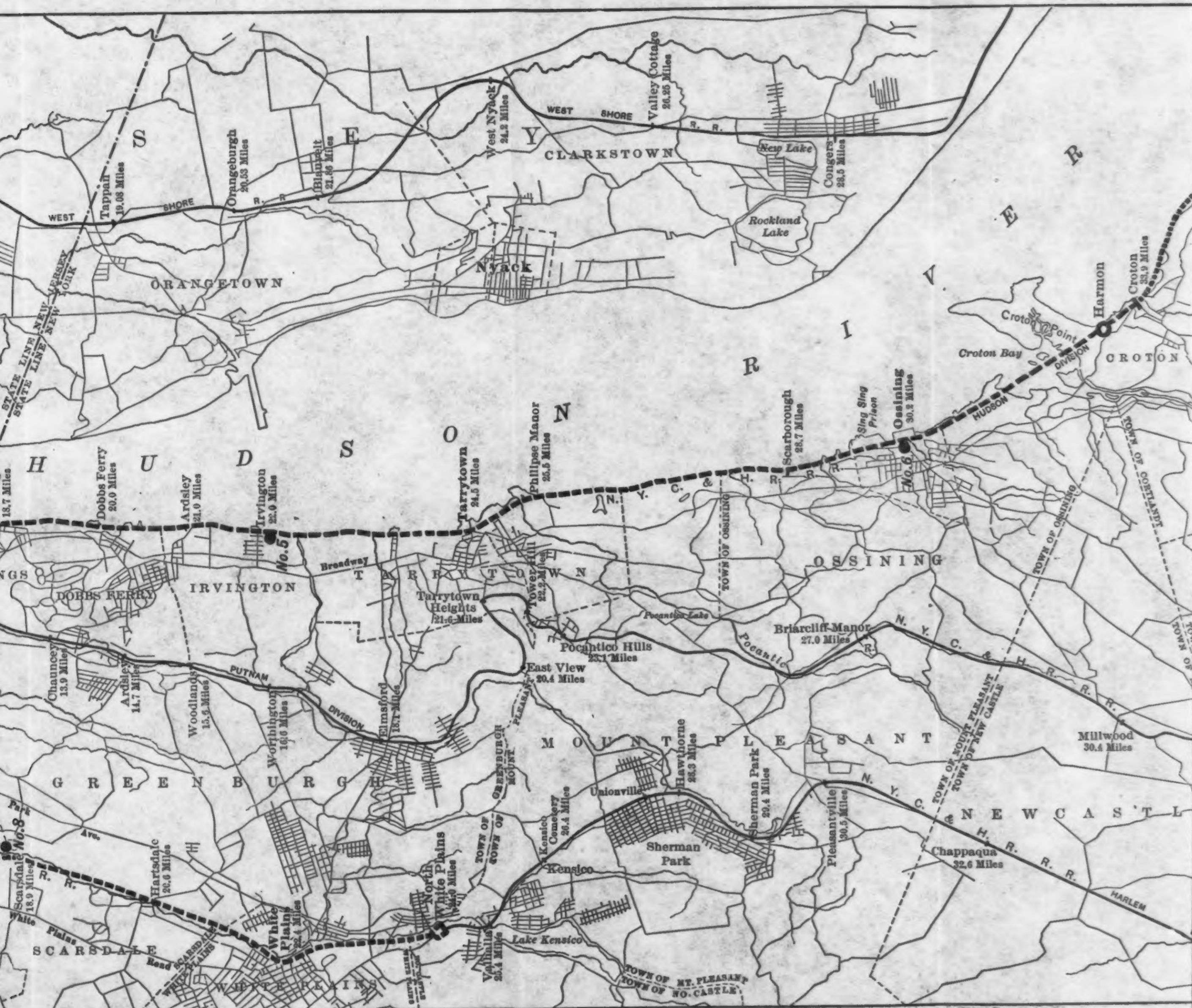
Immediately north of that point is Mott Haven Junction, where the line splits, one leg known as the Harlem Division continuing north



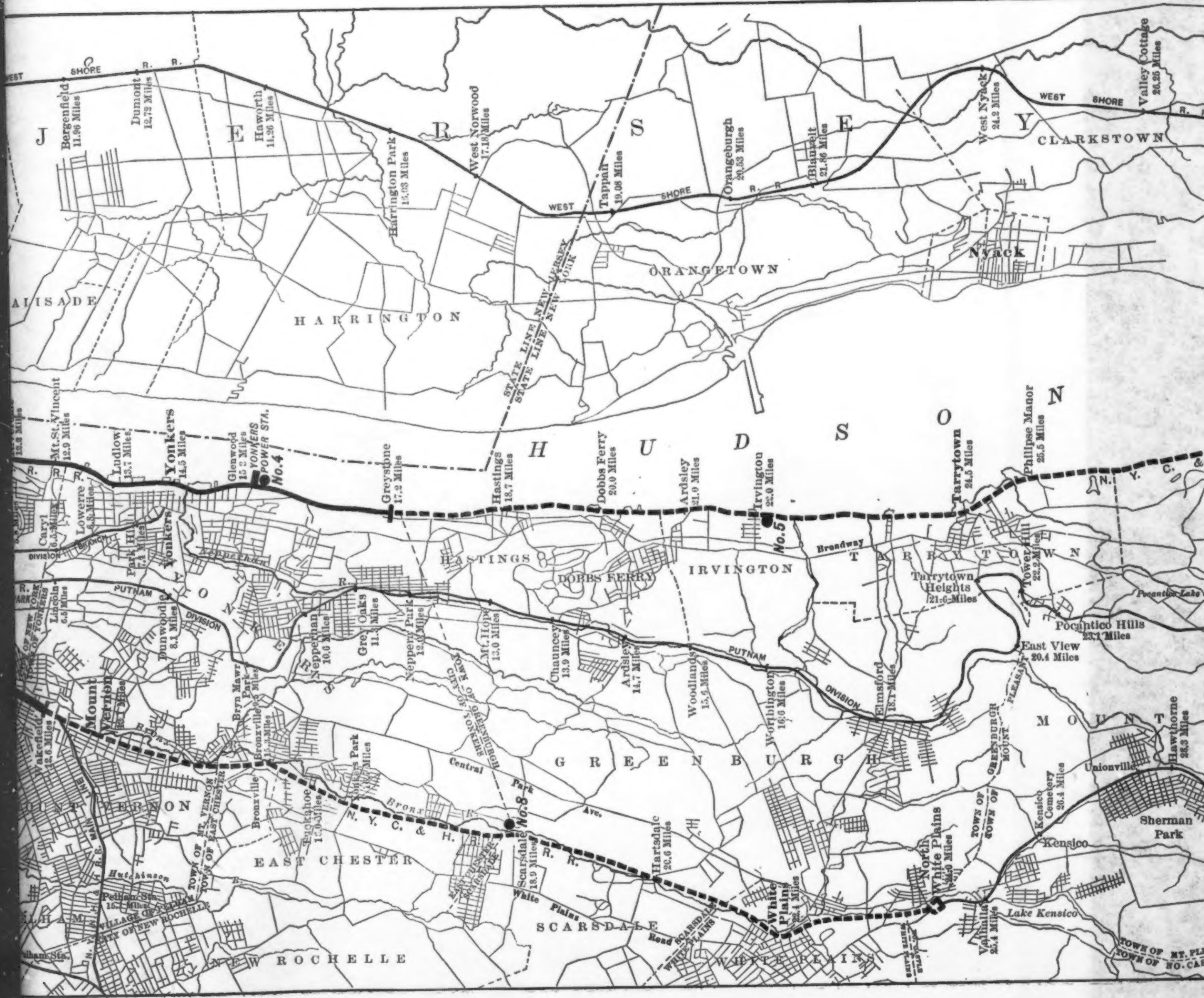
N. Y. C. & H. R. R. R.
ELECTRIC ZONE
NEW YORK CITY AND VICINITY
KEY
INITIAL ELECTRIC ZONE.
PROPOSED IMMEDIATE EXTENSION TO YONKERS.
PROPOSED EXTENSION TO SOUTH CROTON AND NORTH WHITE PLAINS WHEN BOARD OF R.R. COMMISSIONERS APPROVE ELIMINATION OF GRADE CROSSINGS.
POSSIBLE EXTENSION TO PEEKSKILL.
SCALE OF MILES
0 ¼ ½ 1 2
KEY
State Line
City and Town Line
Elevated Railroad System
Subway System
Sub-Station
No.1
JERSEY CITY
Hoboken
New Durham 1.51 Miles
Granton 2.59 Miles
WEEHAWKEN
Weehawken Power Station
WEST SHORE R. R.
Little Ferry
Ridgefield Park
Teaneck 9.15 Miles
West Englewood 10.14 Miles
Bergenfield 11.96 Miles
Dumont 12.72 Miles
Haworth
RIDGEFIELD
ENGLEWOOD
PALISADE
STATE LINE NEW JERSEY
STATE LINE NEW YORK
The Battery
TUNNEL
EAST RIVER
Wallabout Bay
Navy Yard
BLACKWELLS ISLAND
CENTRAL PARK
BROOKLYN
WARDS ISLAND
RANDALLS ISLAND
RIKERS ISLAND
Bowery Bay
North Beach
Flushing Bay
College Point
Flushing
EAST RIVER
N. Y. C. & H. R. R. R.
Broadway
Spuyten Duyvil 10.4 Miles
Kingsbridge
Riverdale 12.8 Miles
Mt. St. Vincent 12.9 Miles
Ludlow 13.7 Miles
Yonkers 14.6 Miles
Glenwood
YONKERS POWER STA.
Van Cortlandt Pk.
Mosholu
Caryl
Lowerre
Park Hill
Dunwoodie
PUTNAM DIVISION
VAN CORTLANDT PARK
Jerome Park Reservoir
Crotona Park
Port Morris
PORT MORRIS POWER STA.
Mott Haven
Tremont
Fordham
Bronx Park
BRONX PARK
Woodlawn Cemetery
Williamsbridge 10.5 Miles
Wakefield
Mount Vernon
MOUNT VERNON
Bronx R.
Van Nest Sta.
Westchester Sta.
Baychester Sta.
Bartow Sta.
PELHAM BAY PARK
PELHAM BAY PARK
Pelham Sta.
Rodman Neck
Eastchester Bay
Bronxville
No.1
No.2
No.3
No.4



WEST SHORE R. R.
Little Ferry 5.01 Miles
Ridgefield Park 6.58 Miles
Teaneck 9.15 Miles
West Englewood 10.14 Miles
Bergenfield 11.96 Miles
Dumont 12.72 Miles
Haworth 14.26 Miles
Harrington Park 15.03 Miles
West Norwood 17.18 Miles
RIDGEFIELD
ENGLEWOOD
PALISADE
HARRINGTON
N. Y. C. & H. R. R. R.
Broadway
Highbridge
Spuyten Duyvil 10.4 Miles
Kingsbridge 9.62 Miles
Riverdale 12.8 Miles
Mt. St. Vincent 12.9 Miles
Ludlow 13.7 Miles
Yonkers 14.5 Miles
Glenwood 15.3 Miles
YONKERS POWER STA. No. 4
Greystone 17.2 Miles
Hastings 18.7 Miles
STATE LINE
HASTINGS
No. 2
No. 3
Harlem River
PUTNAM DIVISION
Van Cortlandt Pk.
Mosholu
Caryl
Lowerre
Park Hill
Yonkers
Jerome Park Reservoir
VAN CORTLANDT PARK
Lincoln
Dunwoodie 8.1 Miles
Nepperhan 10.0 Miles
Grey Oaks
Nepperhan Park 12.0 Miles
Mt. Hope 13.0 Miles
Crotona Park
Morrisania 6.7 Miles
Tremont 7.8 Miles
Fordham
183rd St.
Bronx Park
Woodlawn Cemetery
BRONX PARK
Bronx R.
PORT MORRIS POWER STA.
Port Morris Sta.
RANDALLS ISLAND
RIKERS ISLAND
Williamsbridge 10.5 Miles
Wakefield 12.0 Miles
Mount Vernon
Van Nest Sta. 5.0 Miles
Westchester Sta. 6.0 Miles
Bronx and Pelham Parkway
Unionport
Edenwald
MOUNT VERNON
Bryn Mawr Park
Bronxville
Tuckahoe 15.0 Miles
Central Park
N. Y. C. & H. R.
No. 8
Scarsdale
EAST CHESTER
TOWN OF MT. VERNON
TOWN OF EAST CHESTER
Hutchinson R.
Pelham Sta.
VILLAGE OF PELHAM
CITY OF NEW ROCHELLE
NEW ROCHELLE
PELHAM
Baychester Sta. 7.4 Miles
PELHAM BAY PARK
Bartow Sta.
Rodman Neck
Eastchester Bay
Pelham Sta.
E R I V E R

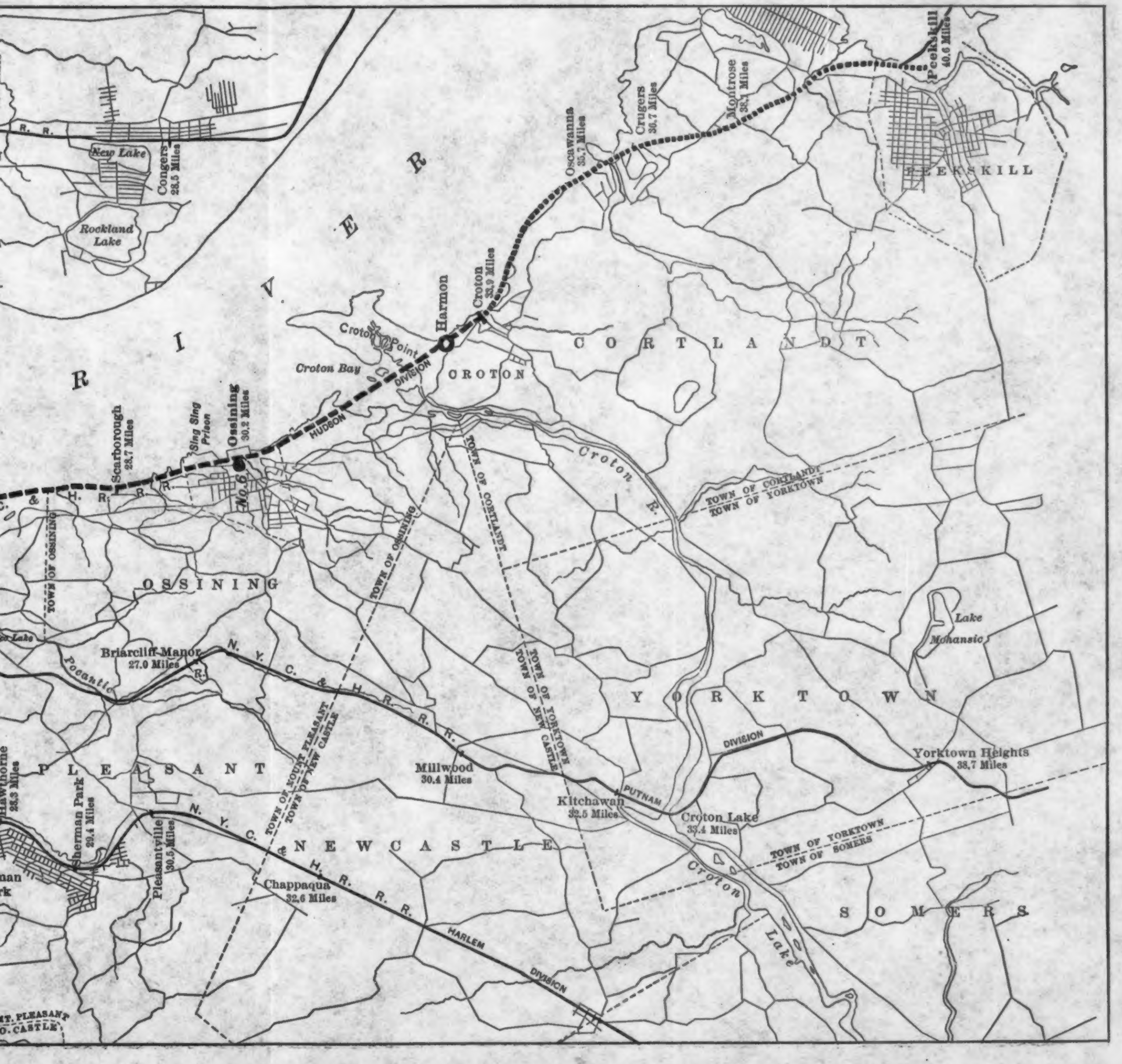


S E Y
CLARKSTOWN
WEST SHORE R. R.
Valley Cottage 26.25 Miles
West Nyack 24.2 Miles
Blauvelt 21.80 Miles
Orangeburgh 20.53 Miles
Tappan 19.08 Miles
New Lake
Congers 28.5 Miles
Rockland Lake
Nyack
ORANGETOWN
STATE LINE NEW JERSEY
STATE LINE NEW YORK
R I V E R
H U D S O N
Harmon
Croton 33.9 Miles
Croton Point
Croton Bay
CROTON
HUDSON DIVISION
Sing Sing Prison
Ossining 30.2 Miles
No. 6
Scarborough 28.7 Miles
Philipse Manor 25.5 Miles
Tarrytown 24.5 Miles
Irvington 22.0 Miles
No. 5
Ardsley 21.0 Miles
Dobbs Ferry 20.0 Miles
18.7 Miles
N. Y. C. & H. R. R. R.
TOWN OF OSSINING
TOWN OF OSSINING
TOWN OF CORTLANDT
OSSINING
Broadway
T A R R Y T O W N
Tower Hill 22.2 Miles
Tarrytown Heights 21.6 Miles
IRVINGTON
DOBBS FERRY
NGS
Pocantico Lake
Pocantico Hills 23.1 Miles
East View 20.4 Miles
Pocantico
Briarcliff Manor 27.0 Miles
N. Y. C. & H. R. R.
Millwood 30.4 Miles
PUTNAM DIVISION
Chauncey 13.9 Miles
Ardsley 14.7 Miles
Woodlands 15.3 Miles
Worthington 16.6 Miles
Elmsford 18.7 Miles
M O U N T P L E A S A N T
TOWN OF MOUNT PLEASANT
TOWN OF NEW CASTLE
Hawthorne 28.3 Miles
Sherman Park 29.4 Miles
Unionville
G R E E N B U R G H
TOWN OF GREENBURGH
TOWN OF MOUNT PLEASANT
Pleasantville 30.5 Miles
N. Y. C. & H. R. R.
N E W C A S T L
Chappaqua 32.6 Miles
HARLEM
Kensico Cemetery 26.4 Miles
Sherman Park
Kensico
Park Ave.
No. 8
Scarsdale 18.9 Miles
Hartsdale 20.6 Miles
R. R.
White Plains Road
North White Plains
White Plains 22.4 Miles
Valhalla 25.4 Miles
Lake Kensico
SCARSDALE
WHITE PLAINS
TOWN OF MT. PLEASANT
TOWN OF NO. CASTLE



J E R S E Y
WEST SHORE R. R.
Bergenfield 11.96 Miles
Dumont 12.72 Miles
Haworth 14.26 Miles
West Norwood 17.18 Miles
Harrington Park 15.53 Miles
Tappan 19.08 Miles
Orangeburgh 20.53 Miles
Blauvelt 21.86 Miles
West Nyack 24.2 Miles
Valley Cottage 26.25 Miles
CLARKSTOWN
Nyack
ORANGETOWN
HARRINGTON
STATE LINE NEW JERSEY
STATE LINE NEW YORK
H U D S O N
Mt. St. Vincent 12.9 Miles
Ludlow 13.7 Miles
Yonkers 14.5 Miles
Glenwood 15.2 Miles
YONKERS POWER STA.
No. 4
Greystone 17.2 Miles
Hastings 18.7 Miles
Dobbs Ferry 20.0 Miles
Ardsley 21.0 Miles
Irvington 22.0 Miles
No. 5
Tarrytown 24.5 Miles
Philipse Manor 25.5 Miles
N. Y. C. &
Caryl
Lowerre
Park Hill 7.4 Miles
Yonkers
HASTINGS
DOBBS FERRY
IRVINGTON
Broadway
T A R R Y T O W N
Tarrytown Heights 21.6 Miles
Tower Hill 22.2 Miles
Pocantico Lake
Pocantico Hills 23.1 Miles
East View 20.4 Miles
PUTNAM DIVISION
Lincoln 6.6 Miles
Dunwoodie 8.1 Miles
Nepperhan 10.6 Miles
Grey Oaks 11.3 Miles
Nepperhan Park 12.0 Miles
Mt. Hope 13.0 Miles
Chauncey 13.9 Miles
Ardsley 14.7 Miles
Woodlands 15.5 Miles
Worthington 16.6 Miles
Elmsford 18.1 Miles
M O U N T
Hawthorne 23.3 Miles
Unionville
Sherman Park
Kensico Cemetery 26.4 Miles
Kensico
Lake Kensico
Valhalla 25.4 Miles
North White Plains 24.0 Miles
White Plains 22.4 Miles
Hartsdale 20.6 Miles
Scarsdale 18.9 Miles
No. 8
Central Park Ave.
G R E E N B U R G H
Mount Vernon
Wakefield 12.6 Miles
MOUNT VERNON
Bryn Mawr Park
Bronxville
Tuckahoe 15.0 Miles
EAST CHESTER
N. Y. C. & H. R. R.
Bronx
SCARSDALE
White Plains Road
WHITE PLAINS
Hutchinson
Pelham Sta.
NEW ROCHELLE

PLATE VII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1079.
WILGUS ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.



to Chatham on the Boston and Albany Railroad, and the other constituting the main line of the Hudson Division, bearing to the west and north, along the banks of the Hudson River, to Albany and beyond. At Woodlawn Junction, on the Harlem Division, is the point of confluence with the New York, New Haven and Hartford Railroad, the very large passenger traffic of which flows over the rails of the New York Central to and from the Grand Central Terminal, a distance of 12 miles.

From the Grand Central Terminal to Woodlawn Junction, the Harlem Division is four-tracked, but for the remainder of the distance within the territory under discussion, but two tracks existed for handling all classes of traffic. Similarly, on the main line, from the junction at Mott Haven, two tracks were called upon to transport both passenger and freight trains, except on the section between Spuyten Duyvil and Scarborough, where a third track aided to some extent.

In addition to the extremely heavy through passenger train service from the New England, northern and western States of the Union, and Canada, there is an important local traffic extending as far out as Harmon, on the Hudson Division, a distance of 33 miles, North White Plains, on the Harlem Division, a distance of 24 miles, and Stamford, on the New York, New Haven and Hartford Railroad, a distance of 34 miles, from the Grand Central Station.

A further burden on the four-track stem between the terminal and Mott Haven Junction is the hauling between those points of "dead" equipment, because of inadequate storage space at the station.

From this recital it will be seen that a termination of the electric zone at the Harlem River, or at Mott Haven Junction just above the river, would entail the stoppage and change of motive power from steam to electricity and *vice versa*, of all kinds of traffic, at a point peculiarly subject to congestion. Moreover, the physical conditions in the neighborhood precluded the construction of the necessary facilities for the storage and care of motive power.

Because of these fundamental objections, and, moreover, guided by the broad-minded policy that growth of traffic responds to the use of electricity, the company decided to extend the limits of the electric zone to the northerly termini of the suburban territory, at Harmon and North White Plains, where ample space is available for loops, yard tracks, and buildings. The geography of the territory is shown on Plate VII.

Reasons for Other Improvements.—This decision, and the demands of growing traffic for more and better facilities, led to the adoption of plans for a new Grand Central Station with two track levels; the separation of track grades and a new overhead eight-track station at Mott Haven; the elimination of all grade, street, and highway crossings; the "four-tracking" of both divisions as far as the termini; many new and enlarged passenger and freight stations; new electric automatic signals, and electric interlocking plants; and many important revisions of alignment and grades.

Reasons for Adopting the Direct-Current System.—About this time the battle had just opened in the United States between the two rival systems of electricity—direct and alternating current. Of course, the advocates of each argued that the other was unsuited to New York Central conditions, and it was only after lengthy and thorough consideration that the direct-current system was selected.

The principal reasons for this conclusion, apart from technical points, may be summarized as insufficient practical development of the alternating-current system for a trunk-line problem requiring absolute reliability of service, restricted clearances which forbade the use of overhead conductors, and legal obstacles to the use of overhead trolley wires carrying high voltages within the limits of the City of New York.

Reasons for Not Using Alternating-Current Equipment on a Direct-Current System.—Some time after this decision had been made, and apparatus had been ordered, the company was urged by outside interests to abandon the type of equipment suited exclusively to the direct-current system, and adopt another type which could operate on both direct and alternating currents. It was claimed that, by making this change, the equipment would be available for use on later extensions of the electric zone where there were no physical or legal objections to the use of alternating current. The wisdom of adhering to the type of equipment already chosen has been proven by recent comparative tests of locomotives of the two types under exactly the same conditions, which demonstrate that the one designed only for direct current consumes from 15 to 25% less current than the one intended for use on both systems. This will effect a saving to the company of at least $140 000 per annum. If to this item is added the economy resulting from less locomotive ton-miles per annum because of the lower weights of locomotive per unit of capacity, and lower wages, fixed charges, and

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FIG. 1.—PORT MORRIS POWER STATION.

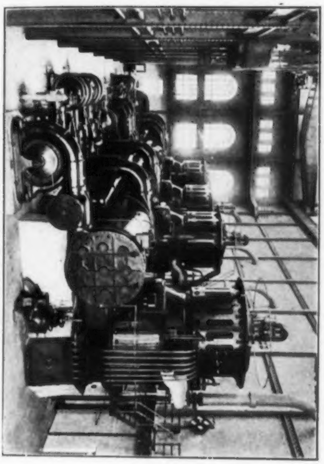

FIG. 2.—INTERIOR, PORT MORRIS POWER STATION.

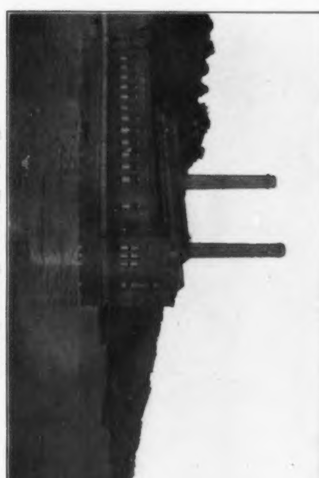

FIG. 3. YONKERS POWER STATION.

FIG. 4.—CABLE TOWER.

maintenance of equipment, because of the smaller number needed to do the same work, the total saving for the ultimate electric zone, resulting from adherence to the adopted type of direct-current locomotive, will be approximately $300 000 per annum.

Reasons for Duplicate Power-Stations and Transmission Lines.—One of the strongest arguments advanced against the substitution of electricity for the well-tried steam locomotive, for the movement of the most important passenger, mail, and express service in the country, is the vulnerability of power-stations and distributing systems to failures of the class which affect, not one, but all, units. To overcome this well-founded criticism, two cross-connected power-stations were decided upon, accessible to both rail and boat coal; and each with sufficient capacity, utilizing its spare unit, and working "overload," to carry the entire demand of the service at the rush hours, should the other fail. It was considered that the growing familiarity of the operating force with the new conditions, and the elimination, in time, of unsuspected defects of installation, would later make the surplus capacity available for other uses, such as increased demands of traffic, the movement of freight trains by electricity, and the operation of the terminals of the company on the west side of Manhattan Island. As a further precaution, duplicate transmission lines were adopted in the more important portions of the territory, so that the failure of one would still leave the other effective for the uninterrupted movement of trains.

Reasons for Storage Batteries.—Even with these two safeguards, there appeared to be vulnerable places, where accidents might put essential features of the service out of commission, and to overcome this, as well as to make suitable regulation of violent fluctuations of load on the power-stations and sub-stations, storage batteries were adopted with capacity sufficient to tide over the usual maximum periods of interruption of current supply, that experience elsewhere has shown may be expected.

Reasons for Combined Locomotive and Multiple-Unit Practice.—While, necessarily, through trains with cars originating at far distant points must be hauled by electric locomotives within the electrified territory, it was evident from the start that, for the company to reap the full advantage from its expenditures, the multiple-unit type of suburban equipment should be adopted that elsewhere had been shown was essential for the propagation of traffic, and the simplification of

operation in congested terminals. By dispensing with locomotives in suburban service, and equipping the individual cars with electric motors controlled from either end of the train, it becomes possible to meet the demand of the public for less interval between trains, and at the same time regulate the cost of operation to the volume of traffic at various periods of the day. The absence of locomotives, and the distribution of power among the cars practically eliminates switching, and movements to and from engine-houses, with a resultant great reduction of the causes that congest terminals. A twofold character of equipment, therefore, was adopted—locomotives for through trains, and multiple-unit cars for the passenger service confined to the electric zone.

Awarding First Contracts.—With all the foregoing questions settled, plans and specifications were actively prepared, and contracts awarded in the fall of 1903 for the apparatus requiring the longest time for delivery, including power-station machinery and locomotives. Later, arrangements were made for the remaining items of the installation, either by contract or by company forces.

General Features of Construction.

Principal Elements of the Installation.—The principal elements of the installation are the duplicate power-stations for generating 3-phase, 11 000-volt, 25-cycle alternating current; the high-tension transmission lines for distributing this current to the sub-stations; the sub-stations for transforming and converting the high-tension alternating current to 660-volt direct current, and for the storage of current in batteries; the direct-current transmission lines for the distribution of energy to the working conductors; the third-rail and overhead conductors at special places, known as working conductors, for the delivery of the 660-volt current to the contact shoes on locomotives and cars; the electrical equipment; repair shops and inspection facilities; interchange terminals for electric and steam power; and last, but not least, the building up of an operating organization to make all this intricate machinery a working success.

Power-Stations.—Each power-station is equipped initially with sixteen water-tube, 625-h.p. boilers, with superheaters and mechanical stokers, and four 5 000-k.w. Curtis turbo-generators, together with the necessary condensers, pumps, exciters, feed-water heaters, and appurtenances. Additional space is provided in the buildings, for a later

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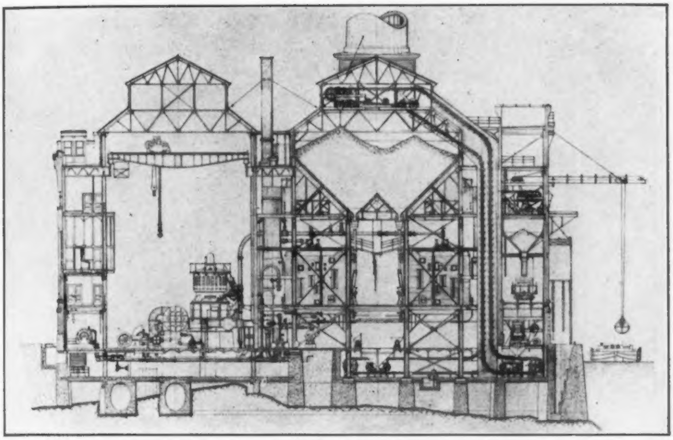

FIG. 1.—TYPICAL CROSS-SECTION OF THE PORT MORRIS POWER STATION.

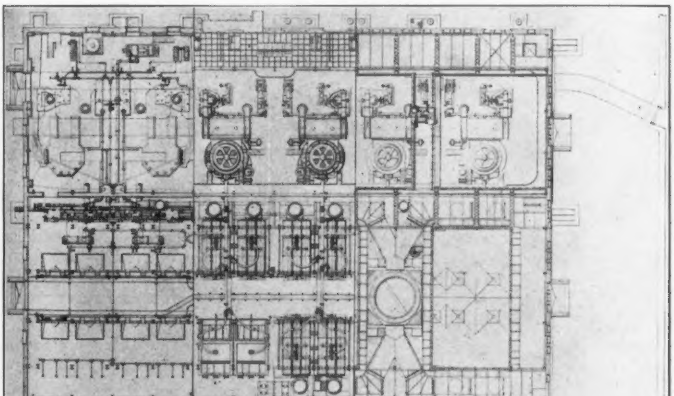

FIG. 2.—TYPICAL PLAN OF PORT MORRIS POWER STATION.

expansion of capacity to the extent of 50% of the initial installation. It will thus be seen that each station has a present normal capacity of approximately 28 000 h.p. (20 000 k.w.), with provision for an ultimate increase to approximately 42 000 h.p. (30 000 k.w.), or a combined ultimate normal capacity of approximately 84 000 h.p. (60 000 k.w.). The two stations are electrically cross-connected, so that, for all practical purposes, they act as one.

Both power-stations are supplied with mechanical plants for transferring coal from car or boat to overhead bins, each station having a storage capacity of 3 500 tons, equal to 9 days' supply under maximum conditions.

A pilot switch-board is located in the gallery of each power-station, but the important control apparatus, including oil-switches, is placed in a separate building, so that serious trouble in the main structure will not disable or injure what may be termed the brains of the system.

The buildings are constructed substantially, of concrete, brick and steel, on stable foundations, and with an architectural treatment suited to the purposes for which they are designed. The twin stacks at each station are of perforated radial brick, have an average internal diameter of 16 ft. 3 in., and rise to a height of 267 ft. above the ground.

A noteworthy fact may be recorded that illustrates one of the advantages of turbo-generators in the economical design of power-station buildings. The capacity required at the Yonkers station is only 110 cu. ft., and that at the Port Morris station 115 cu. ft. per k.w., as compared with from 170 to 255 cu. ft. at the more important reciprocating-engine plants in New York City.

The maximum calculated 4-min. peak load on both stations is 24 000 k.w., at which time 38 trains, of varying speeds, weighing in all 9 800 tons, are assumed to be in motion. The annual output is expected to aggregate 121 000 000 kw-hr., of which 107 000 000 kw-hr. are for the propulsion load and the remainder for lighting and other purposes. These figures do not include the future additional requirements for switching at various yards, the movement of freight trains, and the operation of labor-saving devices at terminals.

11 000-Volt Transmission Line.—The 11 000-volt alternating current from the power-stations is led to the sub-stations by duplicate systems of insulated copper cables in ducts within the populous districts of the city; and by bare copper cables suspended on substantial steel poles set

in concrete bases in the less densely settled districts. This arrangement was adopted only after an exhaustive investigation of line construction throughout the country had proven the greater safety and reliability of well-built aerial wires, where the population is sparse and the line is located on private right of way.

Where the cables pass from one type of construction to the other, they are led through brick towers equipped with lightning arresters.

Owing to the failure of the city to grant the right to place the cables beneath the surface of neighboring streets, it was necessary to locate them within the right-of-way limits of the company, and this required many varied types of construction, often taxing the ingenuity of the engineers to place the conduit pipes where they would be safe from injury. A few of these conditions are illustrated in the accompanying photographs. Altogether, there will be 16 miles of conduit territory, and 46 miles of pole lines, together with 383 splicing chambers.

Sub-stations.—There are to be eight sub-stations, four of which are now in operation. Their total normal rotary capacity will be 27 000 k.w.

Each station contains transformers for reducing the voltage from 11 000 volts primary to 450 volts secondary, and rotary converters for changing the current from alternating to direct at 660 volts. Storage batteries, "floating on the line," are also provided, to regulate the sharp fluctuations of the peculiarly severe short-period demands incident to heavy traction service, and to safeguard the continuity of traffic should perchance the supply of current be interrupted by power-station or distributing failures. This insurance of reliability of service has already demonstrated the wisdom of its adoption. The aggregate momentary capacity of the batteries will be 37 786 k.w., with an hourly capacity of 12 595 k.w.

660-Volt Feeder System.—The 660-volt direct-current system, for conveying energy to the working conductors, consists of copper cables protected and arranged similarly to the high-tension lines already described.

Working Conductors.—The working conductors deliver 660-volt current to locomotives and cars. Third-rail is used at all points, except where intricate switch lay-outs prohibit a continuous conductor near the level of the track. At such places overhead conductors are used.

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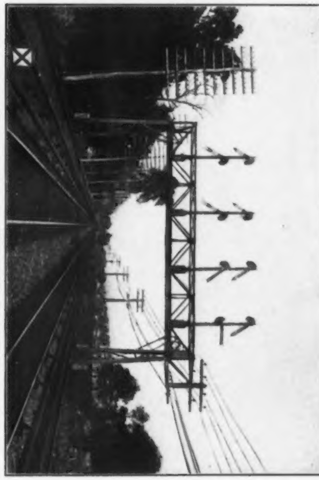

FIG. 1.—AERIAL TRANSMISSION LINE.

FIG. 2.—DUCT TRANSMISSION LINE ON VIADUCTS.

FIG. 3.—DUCT TRANSMISSION LINE ON RETAINING WALLS.

FIG. 4.—DUCT TRANSMISSION LINE IN PARK AVENUE TUNNEL.

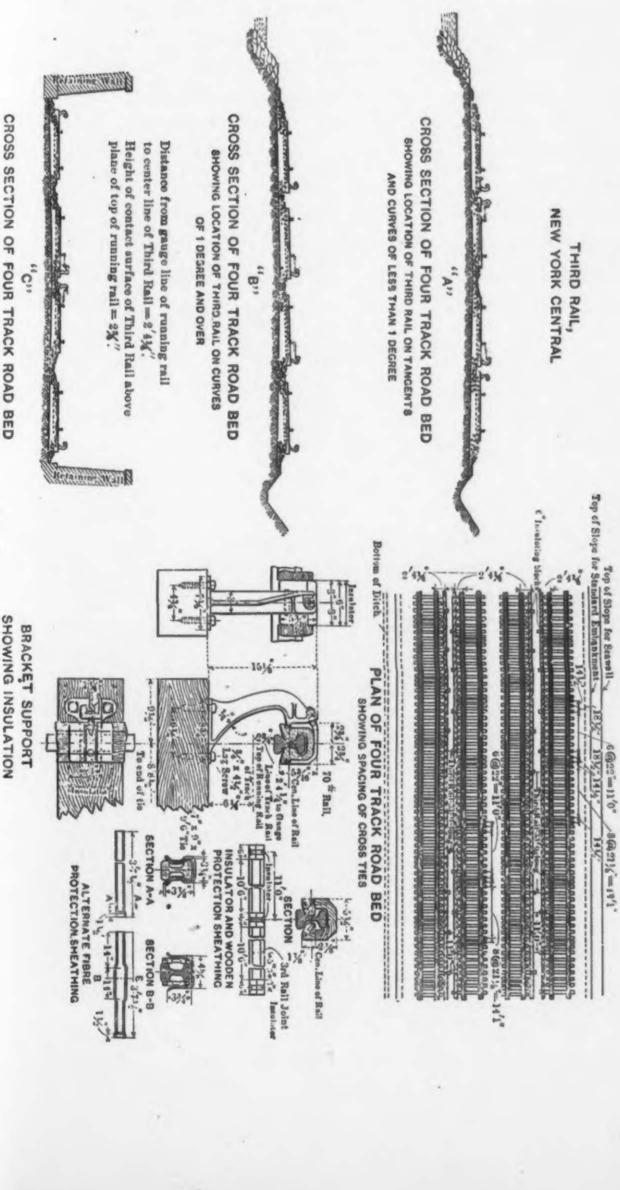
THIRD RAIL,
NEW YORK CENTRAL
"A"
CROSS SECTION OF FOUR TRACK ROAD BED
SHOWING LOCATION OF THIRD RAIL ON TANGENTS
AND CURVES OF LESS THAN 1 DEGREE
"B"
CROSS SECTION OF FOUR TRACK ROAD BED
SHOWING LOCATION OF THIRD RAIL ON CURVES
OF 1 DEGREE AND OVER
Distance from gauge line of running rail
to center line of Third Rail = 2′ 4¾″.
Height of contact surface of Third Rail above
plane of top of running rail = 2¾″.
Retaining Wall
"C"
CROSS SECTION OF FOUR TRACK ROAD BED
SHOWING LOCATION OF THIRD RAIL ADJACENT
TO LONG RETAINING WALLS
PLAN OF FOUR TRACK ROAD BED
SHOWING SPACING OF CROSS TIES
Top of Slope for Seawall
Top of Slope for Standard Embankment
Bottom of Ditch
BRACKET SUPPORT
SHOWING INSULATION
70 # Rail
SECTION
3rd Rail Joint
Insulator
INSULATOR AND WOODEN
PROTECTION SHEATHING
SECTION A-A
SECTION B-B
ALTERNATE FIBRE
PROTECTION SHEATHING

FIG. 1.

either of a temporary character where future track changes are contemplated, or permanently suspended from overhead bridges and buildings. The adopted type of third-rail is unique, for the reason that the current is collected from beneath instead of from the top. This permits the sides and upper parts of the rail to be sheathed in wood or other insulating material in a way that safeguards employees and others from accidental contact, and protects the contact surface from sleet and snow which, with the usual types of top-contact rail, so frequently cause tie-ups of traffic. The manner of construction is such as to secure all these advantages, without encroachment within the clearance lines of the steam equipment, and without precluding the interchange of electric equipment with other lines already using the top-contact type.

At frequent intervals, the direct-current cables pass through small circuit-breaker houses, in which circuit-breakers automatically open and interrupt the flow of current, when, because of accident or injury, there is an improper leak in the third-rail system or the direct-current feeder system. This safety device, therefore, automatically checks the delivery of current to the working conductors, when a continuation of the supply might be disastrous. The circuit-breakers are controlled by cables connected with neighboring sub-stations. Numerous other precautionary measures have been taken for shutting off power promptly in case of accident, such as, for instance, continuous indicator wires for each of the four tracks in the Park Avenue Tunnel, that enable the power to be shut off immediately on any desired track.

In all, there will be 52 miles of territory, embracing 285 miles of track equipped with third-rail, of which more than one-third is completed and in use.

Track Bonds.—The bonding of the track rails for the return current was a task of considerable proportions, because of the intimate relation of the work to traffic. Several ingenious devices were used in expediting the drilling of rails and placing the bonds. The concealed type of bond was used as a protection against the thefts that embarrass traffic and entail pecuniary loss, and to obviate injury by trackmen.

Electrical Equipment.—The electrical equipment now in use comprises 35 locomotives and 180 suburban cars. Of the cars, 125 are equipped with motors. The remainder, for the present, act as trailers, although motors will be added when the electrical service is extended

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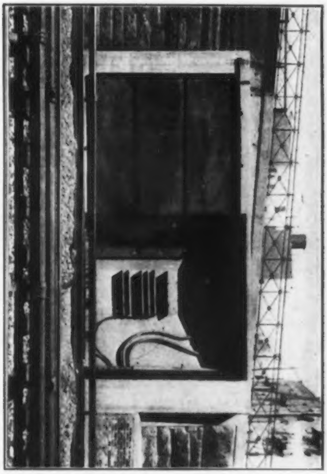

FIG. 1.—TYPICAL SPLICING CHAMBER.

FIG. 2.—SUBMARINE CROSSING, HARLEM RIVER.

FIG. 3.—TYPICAL SUB-STATION.

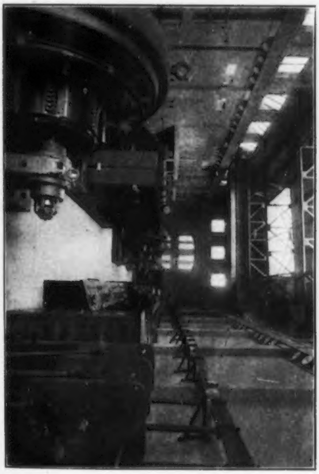

FIG. 4.—INTERIOR OF SUB-STATION.

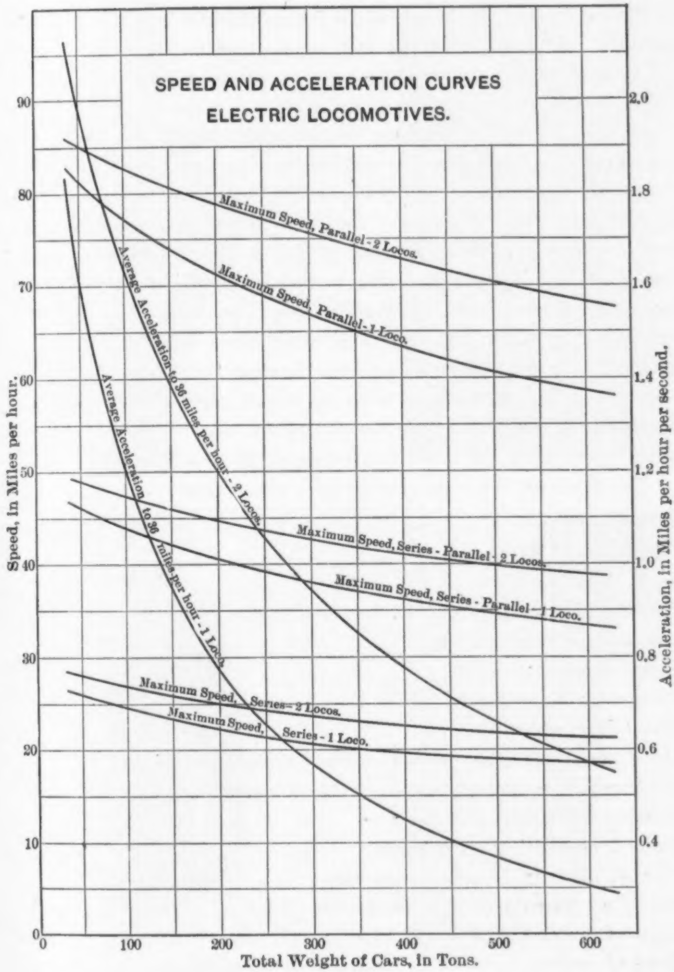
SPEED AND ACCELERATION CURVES
ELECTRIC LOCOMOTIVES.
Maximum Speed, Parallel - 2 Locos.
Maximum Speed, Parallel - 1 Loco.
Average Acceleration to 36 miles per hour - 2 Locos.
Average Acceleration to 36 miles per hour - 1 Loco.
Maximum Speed, Series - Parallel - 2 Locos.
Maximum Speed, Series - Parallel - 1 Loco.
Maximum Speed, Series - 2 Locos.
Maximum Speed, Series - 1 Loco.
Speed, in Miles per hour.
Acceleration, in Miles per hour per second.
Total Weight of Cars, in Tons.
0
10
20
30
40
50
60
70
80
90
100
200
300
400
500
600
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0

FIG. 2.

the full distance to Harmon and North White Plains. The aggregate normal rating of both classes of equipment is 127 000 h.p.

Locomotives.—The locomotive is a peculiarly efficient and powerful machine. Although weighing 94.5 tons, complete, as compared with the 171-ton weight of the heaviest steam passenger locomotives in use by the company, its normal rating of 2 200 h.p. is practically twice that of its rival; it has $76\frac{1}{2}$ tons less weight to haul about, thus effecting a saving of 45% for energy in moving dead tonnage; its concentrated weight per driving axle, 34 250 lb., is 27% less than that of the steam locomotive, without decreasing the total driver weight available for traction; it is capable of running at will in either direction, without the delays and expense of going to the turn-table; it occupies little more than half the track space of the steam locomotive—an important advantage in terminals—and it is much more quickly started and stopped. These advantages have been demonstrated strikingly in practice, both in comparative trials on the 6-mile experimental track near Schenectady, where all the new equipment was tested exhaustively before acceptance, and in regular service in the New York zone.

The principal characteristics of the locomotive are:

Length over all	37 ft. 0 in.
Rigid wheel base	13 " 0 "
Total wheel base	27 " 0 "
Diameter of drivers	44 "
Diameter of truck wheels	$36\frac{1}{2}$ "
Total weight	$94\frac{1}{2}$ tons.
Weight on four drivers	$68\frac{1}{2}$ "
Weight on two trucks	26 "
Horse-power per ton of weight—normal capacity	23
Horse-power per ton of weight—overload capacity	35
Number of motors	4
Normal capacity of each motor	550 h.p.
Normal capacity of each locomotive	2 200 "
Over-load capacity of each locomotive	3 300 "
Type of motors	Gearless, bi-polar.
Type of control	Sprague-General Electric multiple-unit.
Type of heaters for train supply	Westinghouse oil-fired.
Air brakes	Westinghouse graduated-release.

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FIG. 1.—VIEW OF COMPLETED THIRD-RAIL.

FIG. 2.—CIRCUIT-BREAKER HOUSE.

FIG. 3.—OPERATION OF THIRD-RAIL IN WINTER.

FIG. 4.—ELECTRIC LOCOMOTIVE.

Cars.—The suburban cars are constructed of steel and other non-inflammable material, and, while simple in design, have all the features conducive to the safety and comfort of the public. Their leading characteristics are as follows:

Length, over all	62 ft. 0 in.
Length of car body	50 " 0 "
Distance between truck centers	38 " 6 "
Distance between axles of motor trucks	7 " 0 "
Distance between axles of trailer trucks	6 " 0 "
Diameter of wheels—motor trucks	36 "
Diameter of wheels—trailer trucks	33 "
Number of motors on each motor truck	2
Normal capacity of each motor	200 h.p.
Normal capacity of motor car	400 "
Total weight of motor car	53 tons.
Total weight of trailer car	44½ "
Total weight of car body	33½ "
Weight per motor car, due to electrical equipment	8½ "
Horse-power (normal capacity) per ton of weight of electrical equipment	47
Seating capacity	64
Heating system	Both steam and electric.
Lighting system	Both electric and Pintsch gas.
Cooling system for summer season	Two 14-in. electric fans.
Type of control	Sprague-General Electric multiple-unit.
Acceleration, in miles per hour per second	1.2

Comparative Train Weights.—The comparative weights of steam and electric trains in the two classes of service, through and suburban, are interesting, as illustrative of the saving in consumption of energy, and therefore in cost of operation, that accompanies the lower electric train weights; and, also, as justifying the adoption of the multiple-unit instead of locomotive practice for suburban operation.

THROUGH SERVICE.

Steam.	Tons.	Electric.	Tons.
Pacific type locomotive	171.0	Electric locomotive	94.5
8 Pullman cars	400.0	8 Pullman cars	400.0
Total	571.0	Total	494.5

Saving in favor of electric traction = 76½ tons = 13 per cent.

SUBURBAN SERVICE.

Average Number of Cars.

Electric Locomotive.	Tons.	Multiple-Unit Cars.	Tons.
Locomotive	94.5		
4½ steel trailer cars.....	200.0	4½ motor cars..........	238.5
Total............	294.5	Total............	238.5

Saving in favor of multiple-unit practice = 56 tons = 19 per cent.

Shops and Inspection Sheds.—The maintenance of electrical equipment in a high degree of efficiency requires suitable repair shops and inspection sheds, located where the dead mileage will be reduced to a minimum. At both Harmon and North White Plains permanent inspection sheds have been built, and at the former point ample modern shop facilities are provided. As the equipment on both divisions is pooled, any car or locomotive needing repairs can be sent while in regular service to either place, without the expense and loss of time incident to special dead movements.

Interchange Terminals.—At North White Plains, the existing steam engine-house plant is to be enlarged when the extension of electric operation requires added facilities for the interchange of power. At Harmon, space has been provided for ample facilities for the same purpose. Owing to the present curtailment of electric operation because of the backwardness of the State in acting on the abolition of grade crossings north of the limits of the City of New York, temporary terminals have been constructed, at High Bridge on the Hudson Division and at Wakefield on the Harlem Division, with convenient yard arrangements and structures for the care and exchange of power.

Operating Organization of Electrical Department.—The success of a new plant of such magnitude, especially when a change from old to new conditions must be effected without embarrassing an enormous passenger traffic, depends very largely on the organization and personnel of the electrical operating force. It was recognized, at an early stage of the work, that the operation and maintenance of the entire installation required to deliver current to equipment, as well as the maintenance of locomotives and cars, should be under the supervision of those responsible for their construction, leaving to the regular steam

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FIG. 1.—MULTIPLE-UNIT TRAIN.

FIG. 2.—FIRST ELECTRIC TRAIN LEAVING HIGH BRIDGE FOR GRAND CENTRAL STATION.

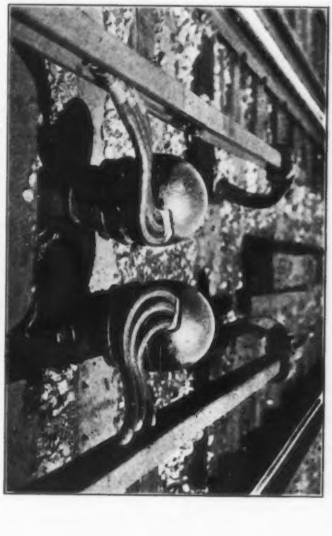

FIG. 3.—"JUMPER" CONNECTION BETWEEN DIRECT-CURRENT FEEDER AND THIRD-RAIL.

FIG. 4.—REACTANCE BOND.

organization the operation of trains with electric current and equipment thus furnished.

As the work on the power-stations, distributing system, and equipment progressed, competent men were gradually employed for inspection and testing purposes, so that, when all was ready for regular operation, there was in existence a skilled, energetic corps of veterans, equal to any emergency, and imbued with a spirit that meant success.

Telephone System.—A word should here be spoken of the independent telephone system which has been constructed for the purpose of bringing all parts of the electric zone in close touch with each other and with the load and train dispatchers.

Other Improvements.—While the principal purpose of this paper is to give an outline of the elements of the electrification of the New York Central suburban zone, it would be incomplete without at least a passing mention of the other important improvements undertaken in conjunction with the change of motive power.

Grand Central Terminal.—Within the territory bounded by Forty-second Street, Fifty-seventh Street, Madison Avenue and Lexington Avenue, the old Grand Central Terminal occupied a parcel of irregular shape, with an area of about 23 acres. The four main tracks from the north descend on grades of from 26 to 53 ft. per mile to the south end of the Park Avenue Tunnel at Fifty-sixth Street; thence they ascend at the rate of 62 ft. per mile in an open cut in the middle of Park Avenue to Fiftieth Street; thence spreading out into the yard, on a slight descent to Forty-fifth Street; and thence on a gentle declivity to the terminal in the train-shed near Forty-third Street. The vital defect of this arrangement was the absence of switching tracks for drilling the yard, north of Fiftieth Street, which necessitated the use of two of the main tracks for that purpose. Consequently, the entrance to the terminal really consisted of but two main tracks for the accommodation of the traffic pouring to and from a four-track line. To increase the congestion, one of these tracks, assigned to drilling service, had also to be used for the storage of steam locomotives at rush hours of the day.

By the use of electricity, it became possible to depress the roadbed south of the low point at Fifty-sixth Street, so as to pass beneath the surface of Park Avenue on either side of the railroad, and thus permit the utilization of the full width of the avenue, 140 ft., without affecting

its use by the public. This gave space for ten instead of four tracks from Fifty-sixth to Fiftieth Streets, of which four are for a legitimate main-line entrance to the enlarged upper yard, two are for drilling the yard, and two on each side, or four in all, are for ingress and egress to the lower-level suburban station. The upper level for through trains will have stub tracks, while the lower level will have a double-track loop at the south end near Forty-third Street.

The depression also admits of the extension of Park Avenue, for its full width, south from Fiftieth to Forty-fifth Streets over the tracks of the yard, and the connection by east and west viaducts of the ends of streets from Forty-fifth to Fifty-sixth Streets, inclusive, now separated by the terminal.

To the 23 acres in the old terminal has been added by purchase 17 acres, making a total area of 40 acres. With the 24 acres obtained by excavating for the suburban station, there will be a total area in the new terminal, when completed, of more than 64 acres. This is equal to an increase over the present space of 178 per cent.

These radical changes make necessary the tearing down of the old station and train-shed, originally built in 1871 and enlarged in 1898 and 1900; and the substitution of a much larger and handsomer structure, suited to the new motive power and more adequate for the proper handling of a rapidly increasing traffic.

It should here be added that electricity brings with it an unexpected boon in the permissible use of overhead spaces termed "air rights," that is denied with steam traction. A vast area in the heart of the greatest city on the continent is thus reclaimed for use as desired for various revenue-producing purposes. In time, this feature will add very largely to the company's assets.

An idea of the difficulties of construction, due to the nature of the underlying material—solid rock—and the necessity of subordinating all efforts to the safe and uninterrupted movement of an exacting and constantly increasing train service, is illustrated in the accompanying photographs.

The magnitude of the new terminal, which has thus to be built while trains and passengers pour in and out, is evident from the quantities of material involved. After being loaded on cars, 3 000 000 cu. yd. of rock and earth are dispatched, at times when the passenger service will permit, to the Hudson Division, for building additional main

FIG. 1.—NORTH WHITE PLAINS TERMINAL.

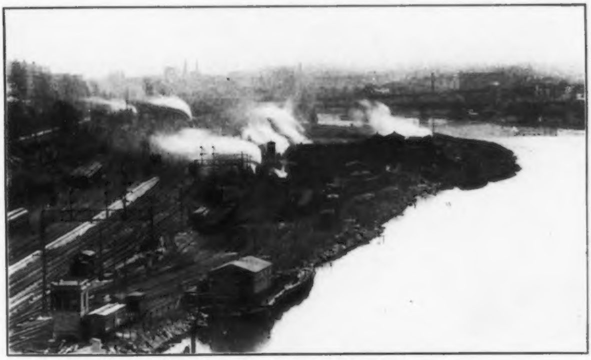

FIG. 2.—HIGH BRIDGE TEMPORARY TERMINAL.

FIG. 3.—WAKEFIELD TEMPORARY TERMINAL.

tracks. In the construction of retaining walls, suburban stations, viaducts, subways, and tunnels, 100 000 tons of steel and 260 000 cu. yd. of concrete are used, in addition to numerous other materials.

The final result will be an electrically operated station and yard with quadruple the capacity of the old one, and with many appurtenances for usefulness and profit, that are additional to the parent purpose of a railroad terminal.

Bronx Improvement.—At Mott Haven Junction, in the Borough of the Bronx, 5.3 miles from the Grand Central Terminal, two four-track lines, making eight tracks in all, merge into the single four-track stem that leads to the terminal. With increased frequency of train service, the grade intersections at such an important junction are inadmissible on the grounds of safety and non-delay to traffic. Therefore, plans have been adopted and work commenced on the raising and lowering of tracks by means of viaducts and tunnels, so as to effect trailing junctions free from grade crossings.

The points of junction are to be moved south about three-quarters of a mile, to the vicinity of the Harlem River; and, near the present connection, on One Hundred and Forty-ninth Street, a new large overhead station is to be built, with eight main tracks. This will permit the abandonment of the old station at One Hundred and Thirty-eighth Street, and remove another of the causes for congestion on the four-track entrance to the Grand Central Terminal. Moreover, this new station will serve the rapidly growing population in the Bronx, which is fast approaching the half-million mark.

Elimination of Grade Crossings.—At the time of the decision to proceed with electric zone improvements, there were, within that territory, forty-four street and highway grade crossings, the abolition of which was deemed precedent to the commencement of electric operation. Of these, one-half were located within the city limits of New York, and these, by agreement with the City, have since been carried over the tracks. None of the remainder has yet been completed, owing to the delay of the State authorities to make effective the provisions of the statute governing grade crossings, and also owing to difficulties in acquiring the necessary additional right of way. However, due to the energetic action of the new Public Service Commission, decisions on many of the crossings have been reached; and the remainder are expected soon. The majority of these eliminations require either

changes of the line of the railroad for considerable distances, as for instance at Mount Vernon and White Plains, or the lifting of the grade of the tracks so that the streets may pass under, as at Yonkers and Tarrytown.

Local Improvements.—The elimination of grade crossings north of New York City, and the growth of business, make obligatory many extensive and costly local improvements, with new passenger and freight stations and yards. The more important ones are at Yonkers, Hastings, Tarrytown, Ossining, and Harmon, on the Hudson Division; and at Mount Vernon, Bronxville, Tuckahoe, and White Plains, on the Harlem Division. Features of the design of these new stations are the avoidance, by means of subways and overhead bridges, of all grade crossings of tracks by passengers; and the placing of the tops of the local platforms on a level with the car floor.

Four-Tracking and Loops.—The anticipated increase in frequency of train service with electric traction, and the urgent necessity of removing causes of congestion in this important entrance to New York City, make mandatory the construction of additional main tracks, so that there will be separate tracks in each direction for high- and slow-speed service; and, where possible, additional tracks for the exclusive movement of freight.

In line with this policy, new main tracks are under construction within the suburban zone, in conjunction with the elimination of grade crossings and improvement of local facilities. The four tracks on the Harlem Division are being extended from Woodlawn Junction to North White Plains, with long middle sidings at frequent intervals, for the passage of passenger trains around freights. The double and triple main tracks on the Hudson Division, as far out as Harmon, are being increased to four, and, at some places, as for instance between Spuyten Duyvil and Yonkers, two additional tracks have been provided for the exclusive use of freight trains. As on the Harlem Division, middle tracks are being built, where needed, for keeping freight trains out of the way of the passenger service.

At Harmon and North White Plains, loops are to be built, for the turning of suburban trains without crossing the express traffic at grade. It will be noted that, with loops at all three termini, and the freedom from grade crossings at Mott Haven Junction, opportunity is given for a constant flow of traffic with an absence of the usual obstructions that cause congestion.

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OF N. Y. C. & H. R. R. R.

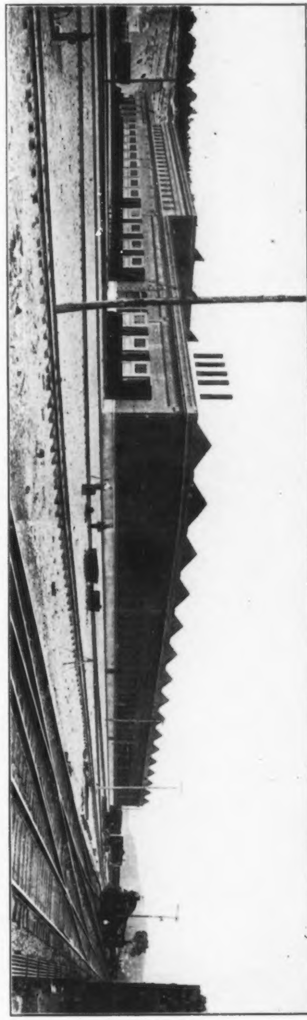


FIG. 1.—HARMON SHOPS.

FIG. 2.—GRAND CENTRAL YARD, LOOKING SOUTH FROM FIFTIETH STREET.

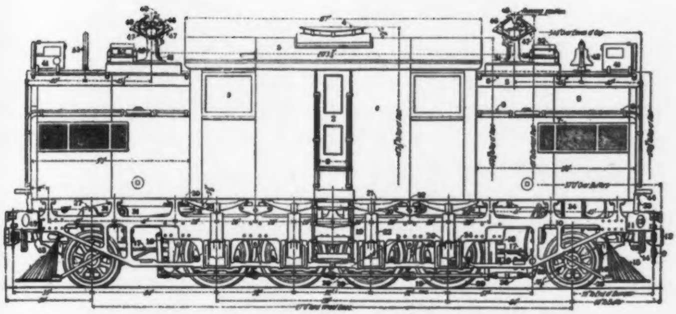


FIG. 3.—ELECTRIC LOCOMOTIVE.

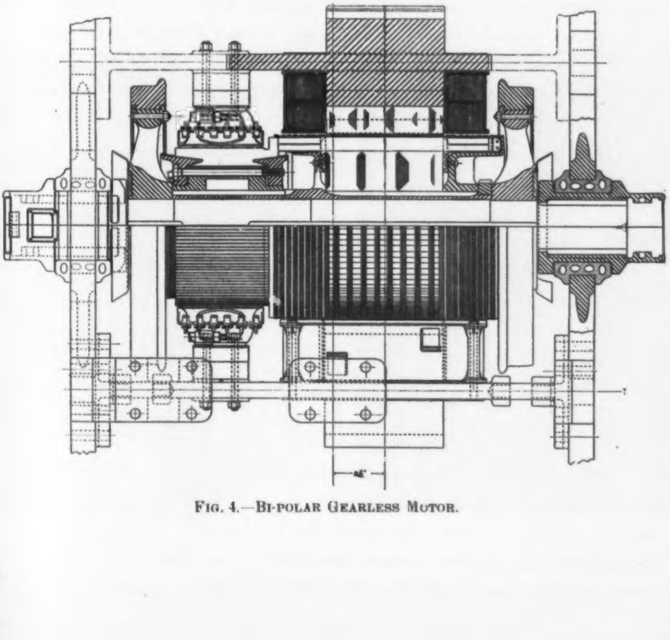


FIG. 4.—BI-POLAR GEARLESS MOTOR.

Increased Capacity of Entrance to Grand Central Terminal.—From the fact that two four-track lines feed into a single four-track stem from Mott Haven Junction to the terminal, the question naturally arises as to what solution the future holds for this restriction on growth of traffic. The present plans of the terminal provide for a future four-track cross-town tunnel connection with the West Side line of the Company, over which the Hudson Division can then enter the terminal without burdening the Harlem Division tracks. This, when built, will afford to the terminal an eight-track entrance connected with both train levels.

Improvements in Alignment and Grades.—In conjunction with these radical changes in the physical condition of the property, it has been considered wise to make at the same time other desirable changes that could not be accomplished later without undue extra cost. At many places, on both divisions, alignment and grades have had careful study, and alterations have been approved which will result in material saving in rise and fall, and in curvature. Many have been completed, and others have been deferred, awaiting the acquisition of right of way and the settlement of legal questions. Among those still in embryo is the improvement between Croton and Peekskill, more than 8 miles in length, which when completed will admit of a still further extension of electric operation.

The advantages to be gained by the principal changes of alignment are as follows:

	Saving in Distance.	Saving in Curvature.
Marble Hill cut-off, including Spuyten Duyvil Tunnel cut-off................	3 944 ft.	137°
Croton to Peekskill....................	4 338 "	333°
Spuyten Duyvil to Mt. St. Vincent.......	9 "	24°
Irvington cut-off.......................	50 "	65°
Totals.	8 341 "	559°

Signals and Interlocking.—Under the old order of affairs, traffic on the Hudson Division from the north ran right-handed to Spuyten Duyvil, where it was transposed to left-handed operation so as to harmonize with the left-handed practice on the Harlem Division. The design of the new Grand Central Terminal and a possible future con-

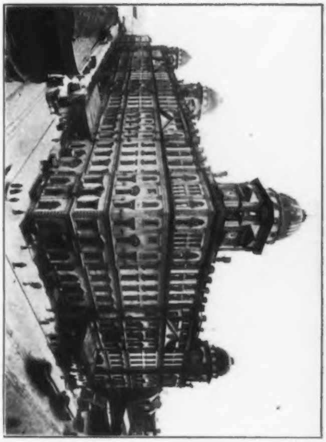


FIG. 1.—PRESENT GRAND CENTRAL STATION.

FIG. 2.—PRESENT GRAND CENTRAL TRAIN-SHED.

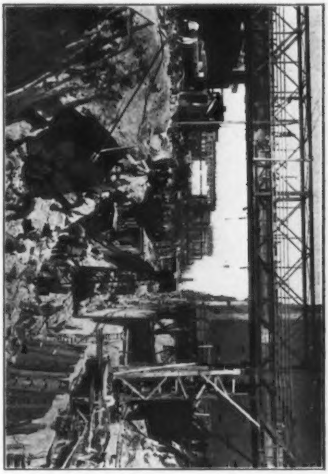


FIG. 3.—EXCAVATION IN PROGRESS, GRAND CENTRAL TERMINAL.

FIG. 4.—EXCAVATION IN PROGRESS, GRAND CENTRAL TERMINAL.

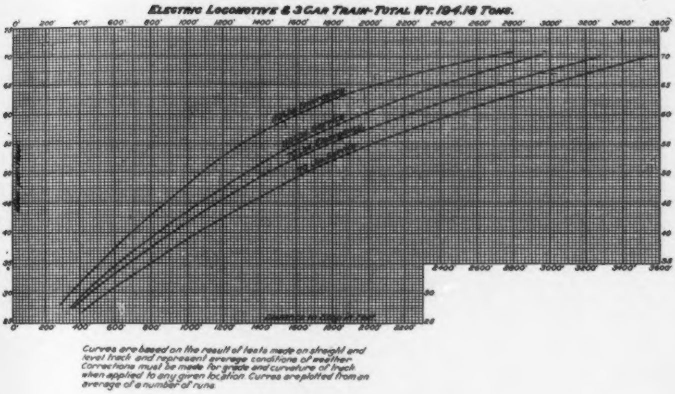


FIG. 5.—TRAIN BRAKING CHART—DISTANCE.

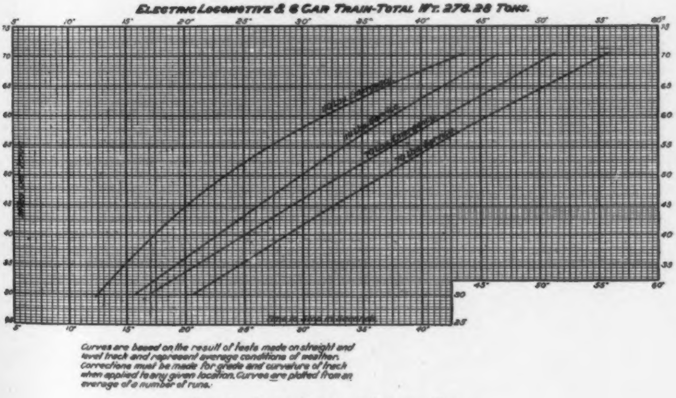


FIG. 6.—TRAIN BRAKING CHART—TIME.

nection with the city subway system, as well as the desirability of avoiding the grade crossing at Spuyten Duyvil, led to the decision to make the right-handed system of operation uniform throughout the suburban zone. In considering the effect of this reversal of traffic on the existing signals and interlocking plants, it was also realized that the controlled-manual system in use on a large portion of the territory was insufficiently elastic for the quick handling of a frequent electric train service on four or more tracks. Accompanying these traffic reasons for radical changes in the old signals and interlockings was the equally important fact that the use of track rails for return propulsion current to the power-stations completely deranged the signal circuits. Then, too, the many additions and changes to tracks made imperative the abandonment of the larger part of the old plants.

All these causes led to the adoption of new electric automatic signals and electric interlocking plants for the entire zone, the predominant feature of which is the reactance bond, which permits the free passage of propulsion current through the track rails, but, where desired, stops the passage of the alternating signal current circuit.

Fences.—One of the worst evils with which American railroads have to contend is trespass. Right of way and tracks are considered public highways, and the petty courts refuse or neglect to impose adequate punishment on those who thus risk their lives in dangerous places. This freedom of use of the railroad's property also leads to thieving which, in the aggregate, causes large losses to the company. The change to electric traction by no means minimizes these evils. More frequent trains, and the presence of electricity, increase the risks, while copper cables and bonds attract the thief. To guard against these increased dangers, the entire electric zone is to be enclosed with man- and boy-proof fences. The portion within the settled districts consists of iron pickets and concrete posts of a pleasing design.

Chronology.—Following the placing of orders in the fall of 1903, work was pushed energetically on all items of construction required for the operation of the initial electric zone south of Wakefield and High Bridge. It should be borne in mind that the larger part of the work had to be performed on or about tracks congested with traffic, which entailed danger to employees, delay to many parts of the work, and expense. It is a pleasure to record, however, that not an accident occurred to regular train service, nor, with a few minor exceptions, any

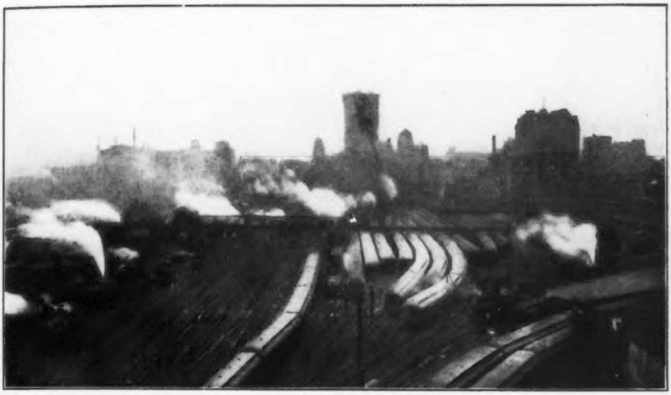


FIG. 1.—BEFORE.

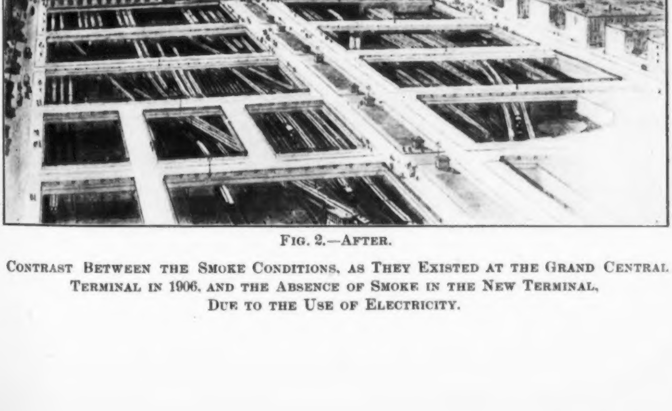


FIG. 2.—AFTER.

CONTRAST BETWEEN THE SMOKE CONDITIONS, AS THEY EXISTED AT THE GRAND CENTRAL TERMINAL IN 1906, AND THE ABSENCE OF SMOKE IN THE NEW TERMINAL, DUE TO THE USE OF ELECTRICITY.

delay to traffic, due to construction. Such accidents and delays as did occur were from other causes.

The following dates mark the progress of the electrical features:

Initial informal test of first electric locomotive	October 27th, 1904.
First formal test of electric locomotive	November 12th, 1904.
Port Morris Power-Station:	
Commencement	May 15th, 1904.
First current	July 1st, 1906.
Transmission Lines:	
Commencement	February 17th, 1905.
Ready for service	September 30th, 1906.
Sub-stations:	
Commencement	July 6th, 1905.
Ready for service	September 30th, 1906.
Working Conductors:	
Commencement	January 2d, 1906.
Ready for service	December 11th, 1906.
Electrical Equipment:	
First operated in New York City	July 20th, 1906.
First train into Grand Central Terminal	September 30th, 1906.
Electrical Operation:	
First schedule multiple-unit train	December 11th, 1906.
First schedule electric locomotive	February 13th, 1907.
First regular shop train	April 14th, 1907.
Completion of change of motive power:	
Schedule trains	July 1st, 1907.
Reversal of traffic	August 25th, 1907.

Because of the burdensome conditions of traffic, and complicated changes in the signal and interlocking systems, about 6 months were thus consumed in making the change of motive power complete, after the first schedule train was operated.

Initial Zone Operation.—As previously stated, the company was forced to confine temporarily the change of motive power to the operation of the suburban zone terminating at High Bridge, 7 miles out; and at Wakefield, 13 miles from the terminal. This postpones for two or three years the extension of electrical service to the northerly termini of the suburban zone. In the meantime, the power on through trains is changed at the temporary termini. At the same points, multiple-unit trains north-bound have steam locomotives attached and thence proceed as non-electric trains; and south-bound the steam locomotives are detached and the trains continue by electricity without locomotives. The average time required for making the changes, including that lost in slowing down and regaining speed is as follows:

Through trains with locomotives........................	4½	min.
Multiple-unit trains, north-bound........................	3	"
Multiple-unit trains, south-bound........................	2½	"

On the Hudson Division this delay has been largely compensated by shortening the line at Marble Hill and the elimination of grade track crossing at Spuyten Duyvil.

RESULTS.

Expectations from Electrification.—Now that the change of motive power in the initial electric zone has been completed for sufficient time to gain at least a preliminary idea of the results, the question naturally arises, with what success has the change met expectations?

It has already been explained that the principal reasons for undertaking the work were twofold:

(1).—Demand of the public for the abolition of the nuisances incident to the use of steam locomotives south of the Harlem River; and

(2).—Need for increased capacity of the terminal, by the elimination of a large proportion of the switching movements required with steam locomotive practice; and relief to the main line entrance to the terminal by reducing its use for haulage of dead locomotives and cars to Mott Haven.

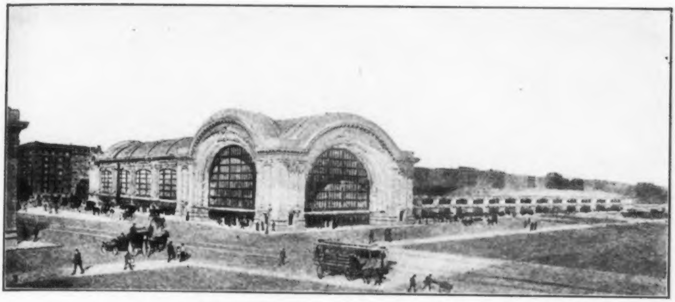


FIG. 1.—PROPOSED BRONX STATION.

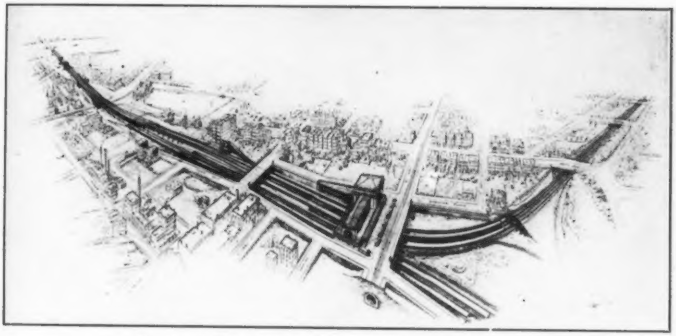


FIG. 2.—ORTHOGRAPHIC VIEW OF BRONX IMPROVEMENT

FIG. 3.—TYPICAL GRADE CROSSING ELIMINATION, WITH OVERHEAD STATION AND TRACKS BENEATH (UNIVERSITY HEIGHTS).

As secondary considerations there were:

(3).—The possibility of sufficient economy in operation at least to offset largely the additional fixed charges on the cost of the electrical installation; and

(4).—Opportunities for an ultimate large increase in traffic and corresponding growth of revenue to justify the expenditure for all improvements within the suburban zone.

What do the observations made thus far disclose?

The first two expectations have been completely realized.

Park Avenue Tunnel.—The atmospheric conditions in the Park Avenue Tunnel show marked improvement, even with the presence of the remaining New Haven Company's steam service.

Increased Terminal Capacity.—The effect on the operating efficiency of the terminal has been very gratifying, the increased capacity being estimated at one-third. There has also been a large reduction in the number of shop or "dead" trains to and from Mott Haven.

Reduced Cost of Operation.—The results, as regards the third expectation, have been most surprising. The operation, for a considerable period, of steam and electric equipment side by side has afforded an unexampled opportunity for a true comparison of costs of operation. Until now, data on this subject have been based on theory, ignoring many of the indeterminate features of actual operation that have such a weighty effect on costs. For instance, among the variables entering into an analysis of this character are:

(a).—Cost and quantity of coal and water at the power-station, and on the steam locomotive tender;

(b).—Relation of ton-mileage of the motive power to total ton-mileage, including motive power and cars;

(c).—Frequency and volume of traffic;

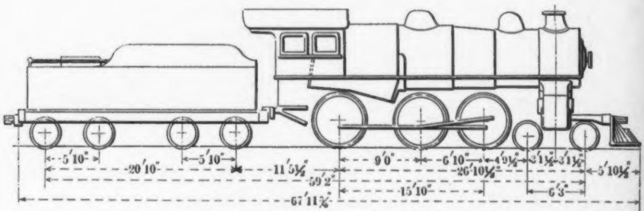
(d).—Mechanical and electrical design of motive power as affecting repairs, and hours available for active service;

(e).—Fixed charges, depreciation, and maintenance on all items of both kinds of service, that have a bearing on comparative results, including land, structures, and equipment.

In other words, to obtain a true comparison, observations must be made under like conditions in a known service.

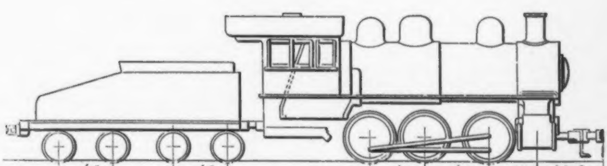
With this object in view, a typical steam switching locomotive, engaged in terminal service, and a steam passenger locomotive, assigned to road service, were each selected for observation in the same class of traffic with electric locomotives. The terminal service embraced switch-

STEAM LOCOMOTIVES USED IN COMPARATIVE TESTS



Weight on drivers, working order .... 156 000 lb.
Weight on truck, working order .... 16 500 lb.
Weight; total of engine .... 191 500 lb.
Weight of tender, loaded .... 118 000 lb.

STEAM LOCOMOTIVE USED IN
ROAD TESTS.
CLASS-F-2-d.
(No.1978.)



Weight on drivers, working order .... 152 500 lb.
Weight, total of engine .... 152 500 lb.
Weight of tender, loaded
4500-gal. tank .... 89 500 lb.
5100-gal. tank .... 91 500 lb.

STEAM LOCOMOTIVE USED IN
SWITCHING AND HAULING TESTS.
CLASS B-10.

FIG. 7.

ing at the Grand Central yard, and hauling dead cars to and from Mott Haven storage yard, a distance of 6 miles. The road service comprised the hauling of schedule trains by the electric locomotive between the Grand Central Terminal and Wakefield, 12½ miles; and

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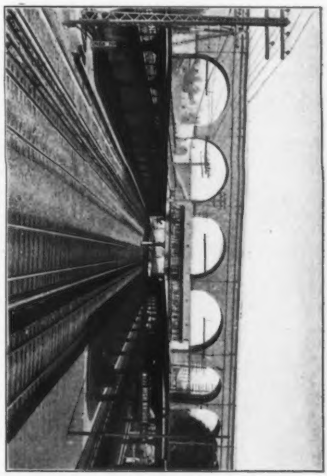

FIG. 1.—TYPICAL GRADE CROSSING ELIMINATION, WITH OVERHEAD STATION AND TRACKS BENEATH (HIGH BRIDGE).

FIG. 2.—MARBLE HILL CUT-OFF, BY WHICH SEVEN GRADE CROSSINGS HAVE BEEN ABOLISHED.

FIG. 3.—YONKERS GRADE CROSSING WORK IN PROGRESS.

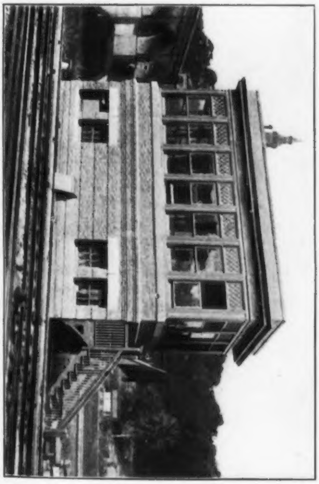

FIG. 4.—TYPICAL SIGNAL TOWER.

the same trains by steam between Wakefield and North White Plains, 11½ miles.

Observers constantly rode the locomotives for the period of the tests, namely, September 12th to 27th, 1907, in terminal service, and October 4th to 18th, 1907, in road service. Cyclometers and wattmeters registered actual distances, speeds, and current consumption. Record was also kept of the number of cars switched and hauled, and the proportion of time each day engaged in actual service, awaiting duty, and laid up for inspection and repairs.

The coal used contained 14 000 B. t. u. per lb., and the cost, per ton of 2 240 lb., was:

Steam locomotive in terminal service (anthracite)....	$5.00 per ton.
Steam locomotive in road service (bituminous)......	3.50 " "
Port Morris power-station (bituminous)............	3.05 " "

Water, per 1 000 gal., cost as follows:

Terminal service and at power-station..................	13⅓ cents.
Road service..	5 "

The cost of electric current, when the power-station designed load is attained, is taken at 2.6 cents per kw-hr., delivered at the contact shoes of the equipment, and includes all operating and maintenance costs, interest on the electrical investment required to produce and deliver the current, depreciation, taxes, insurance, and transmission losses. The details of this cost are:

Items.	Operating costs.	Fixed charges.	Total.
Power-station....................	0.58 cents.	0.44 cents.	1.02 cents.
Transmission losses..............	0.19 "	0.15 "	0.34 "
Distributing system and sub-stations..........................	0.32 "	0.92 "	1.24 "
Totals........................	1.09 cents.	1.51 cents.	2.60 cents.

Locomotive wages are practically identical for each class of service.

Table 1 shows the details of locomotive repairs, maintenance, and fixed charges for each class of service, from which it will be noted that, although the fixed charges and depreciation of the electric loco-

TABLE 1.—COMPARISON OF COSTS PER DAY OF AVAILABLE SERVICE OF STEAM AND ELECTRIC LOCOMOTIVES FOR INTEREST, DEPRECIATION, REPAIRS, INSPECTION, AND HANDLING.

SUBJECT.	STEAM.				ELECTRIC.			
	Description.	Amount per annum.		Per day.	Description.	Amount per annum.		Per day.
Interest	4¼% on $15 000		$637.50		4¼% on $30 000		$1 275	
Depreciation	5% on $15 000		750.00		5% on $30 000		1 500	
Repairs	General at West Albany	$1 170			General at Harmon	$468		
	Running at Mott Haven	414			Running at High Bridge and Wakefield	166		
	Trips to shops, 300 miles	168			Trips to shops, 60 miles	34		
	Use of shops	90	1 842.00		Use of shops	36	704	
	Total for 335 days available for service.		$3 229.50	$9.64	Total for 350 days available for service.		$3 479	$9.94
Handling and inspection, including fixed charges and maintenance of land and structures	Mott Haven engine-house plant, 365 days		1 231.00	3.37	High Bridge and Wakefield inspection sneds, 365 days		200	0.55
Total			$4 460.50	$13.01			$3 679	$10.49

The saving in favor of the electric locomotive, therefore, is $2.52 per day, equal to 19 per cent.

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FIG. 1.—SIX-TRACKING, HUDSON DIVISION.

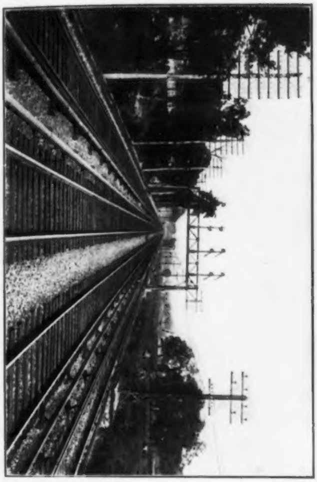


FIG. 2.—FOUR-TRACKING, HARLEM DIVISION.

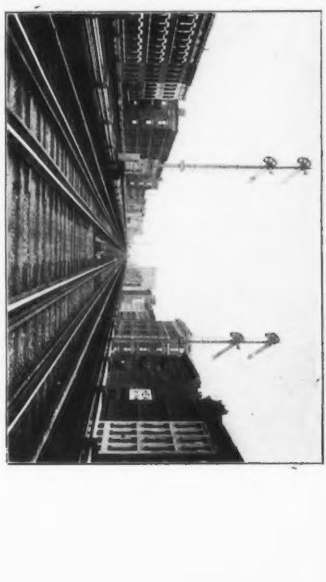


FIG. 3.—TYPICAL SIGNALS, PARK AVENUE VIADUCT.

FIG. 4.—TYPICAL FENCING, NEW YORK CITY

motive are higher than those of the steam, owing to the greater first cost, the net result is in favor of the electric locomotive, due to lower costs for repairs and maintenance. These results are based on actual observations of the steam locomotive covering a period of several years; and of the electric locomotive for two years on the experimental track near Schenectady and one year in the New York zone. The reasons for the lower cost of repairs on the electric machine are the simplicity of construction and the minimum number of mechanical parts. It is also worthy of comment that the electric locomotive costs very much less per day for repairs and maintenance, due to lower expenses for land and structures, and fewer days out of service. For instance, the fixed charges and cost of maintenance and operation of the extensive steam engine plant on costly land, are comparable with the simple inspection-shed charges of the electric locomotive.

The Schenectady experiments indicated that the cost of repairs of the electric locomotive of this type is about two-fifths of that of the steam locomotive of a corresponding age and capacity.

The results of these observations are shown in detail in Table 2, Plate XXI, and are summarized in Table 3. They show that, under the stated conditions, the electric locomotive has the following advantages over its steam rival:

19% saving in locomotive repairs and fixed charges.
18% saving in dead time for repairs and inspection.
25% greater daily ton-mileage.
6% saving in locomotive ton-mileage in hauling service.
11% saving in locomotive ton-mileage in switching service.
16% saving in locomotive ton-mileage in road service.
12% net saving in cost in hauling service.
21% net saving in cost in switching service.
27% net saving in cost in road service.

Even better results may be expected during winter months, when steam locomotives are subjected to many conditions that cause additional expenses not incident to the electric locomotive.

Reduced Cost of Grand Central Terminal Operation.—Owing to the partial use of steam switching locomotives, and the presence of the New Haven Company's steam road locomotives at the terminal, the full benefits of change of motive power have not yet been secured.

TABLE 3.—SUMMARY OF COMPARATIVE TESTS OF STEAM AND ELECTRIC LOCOMOTIVES.

Kind of locomotive.	Miles per day.	Cars per day.	Busy hours per day.	Hours ready for duty daily.	Percentage of time dead.	Total ton-miles daily.	Car ton-miles daily.	Percentage of car ton-miles to total.	Car ton-miles per busy hour.	Coal or current per car ton-mile.	Total cost per car, in cents.	Speed and stops. Average miles per hour.	Maximum miles per hour.	Stops.	Cost per 1 000 car ton-miles. Supplies.	Wages.	Interest, depreciation, and repairs on locomotives.	Total.	Watt-hours required to do work of 1 lb. coal.
SWITCHING SERVICE—GRAND CENTRAL TERMINAL.†																			
Steam	10.91	55	†1.83	†6.16	†0.52	2 580	916	0.35	501	3.36 lb. coal.	35.2	....	....	....	$8.06	$5.34	$7.61	$21.01	
Electric	11.13	53	†2.01	†6.80	†0.26	1 980	914	0.46	445	264 watt-hr.	28.5	....	....	....	6.88	5.25	4.40	16.53	79
Advantages in favor of electric locomotives.	0.22	....	†0.18	†0.64	†0.26	......	......	0.11	......	....	6.7	....	....	....	1.18	0.09	3.21	4.48	
HAULING TO AND FROM MOTT HAVEN.†																			
Steam	40.0	45	†3.36	†5.18	†0.53	16 540	11 720	0.71	3 490	0.46 lb. coal.	51.6	11.9	48	0.9	1.12	0.35	0.52	1.99	
Electric	78.4	95	†6.41	†10.42	†0.30	30 370	23 310	0.77	3 640	44.3 watt-hr.	43.2	12.3	45	2.0	1.16	0.31	0.28	1.75	96
Advantages in favor of electric locomotives.	38.4	50	†3.05	†5.24	†0.23	13 830	11 590	0.06	150	....	8.4	0.4	3	1.1	......	......	0.24	0.24	
ROAD SERVICE.*																			
Steam	74.04	28	3.72	‡11.11	‡0.54	25 620	12 660	0.49	3 400	1.22 lb. coal.	126.0	19.9	60	8.6	2.03	0.28	0.46	2.77	
Electric	126.22	43	5.34	‡13.70	‡0.43	33 210	21 510	0.65	4 080	52.3 watt-hr.	100.0	23.6	55	2.9	1.37	0.31	0.34	2.02	43
Advantages in favor of electric locomotives.	52.18	15	1.62	‡2.59	‡0.09	7 590	8 850	0.16	630	....	26.0	3.7	5	....	0.66	......	0.12	0.75	

* Portion of time of locomotives engaged in other service not shown in this table. † Switching and hauling done by same locomotives.
‡ Total time of locomotives in all classes of service.

Items.	Switching.	Hauling.	Total.
Pounds of coal per ton-mile of cars....	3.36	0.46	0.67
Gallons of water per ton-mile of cars..	3.39	0.47	0.69
Watt-hours per ton-mile of cars........	3.64	44.3	32.6

TABLE 2.—COMPARATIVE TESTS OF STEAM AND ELECTRIC LOCOMOTIVES IN SWITCHING AND HAULING SERVICE:

GRAND CENTRAL TERMINAL, N. Y. RUNS MADE BETWEEN SEPTEMBER 12TH AND 27TH, 1907.

SWITCHING, GRAND CENTRAL STATION.

No. of test.	Kind of Power.	Hours. Busy.	Idle. Waiting.	Idle. Dead.	Idle. Total.	Miles run.	Cars handled.	Ton-miles. Total.	Ton-miles. Cars only.	Ton-miles. Per busy hour.	Supplies. Coal. Tons.	Supplies. Coal. Pounds, ton-miles.
1	Steam....	1.69	6.65	12.45	19.10	11.66	29	1 962	540	320	1.25	1.42
2	Electric..	7.63	13.25	3.12	16.37	46.29	306	9 542	5 098	669	......	......
3	Steam....	1.80	3.12	6.91	10.03	12.63	22	6 994	517	286	2.1	0.67
4	Electric..	0.54	1.22	2.95	4.17	3.38	16	933	612	1 133	......	......
5	Steam....	0.79	1.92	3.31	5.23	5.17	40	1 141	523	663	0.84	1.20
6	Electric..	1.02	4.06	5.31	9.39	5.69	113	651	120	118	......	......
7	Steam....	2.65	6.51	4.54	11.05	13.61	32	4 130	2 562	969	1.84	1.00
8	Electric..	1.44	5.43	4.84	10.27	7.73	33	1 049	315	219	......	......
9	Steam....	1.12	2.52	6.64	9.16	5.28	20	1 116	482	430	0.88	1.17
10	Electric..	0.82	2.68	5.28	7.96	4.67	29	734	269	328	......	......
11	Steam....	1.34	4.06	8.86	12.92	8.81	36	1 720	680	507	1.20	1.56
12	Electric..	0.15	0.30	0.30	0.60	0.72	10	169	100	666	......	......
13	Steam....	5.97	10.13	2.15	12.28	37.53	157	7 283	2 796	468	4.87	1.41
14	Electric..	2.35	5.53	1.05	6.58	12.84	46	1 954	739	314	......	......
15	Steam....	1.42	2.82	5.76	8.58	6.60	41	1 427	610	430	0.88	1.33
16	Electric..	1.88	4.07	0.71	4.78	9.96	45	1 846	905	482	......	......
17	Steam....	1.75	2.50	5.46	7.96	7.68	23	1 186	248	142	0.75	1.42
18	Electric..	1.05	3.08	4.55	7.63	4.77	19	777	324	309	......	......
19	Steam....	1.61	6.01	4.48	10.49	9.69	46	1 711	574	356	1.00	1.42
20	Electric..	1.58	3.92	0.96	4.88	8.27	53	1 457	662	419	......	......
21	Steam....	1.17	3.05	6.91	9.96	7.61	31	1 440	535	457	1.04	1.61
22	Electric..	2.51	6.67	1.86	8.53	12.77	37	2 318	967	373	......	......
23	Steam....	2.50	3.92	4.13	8.05	15.46	106	3 545	1 685	674	1.97	1.86
24	Electric..	1.99	5.88	0.97	6.85	11.50	24	1 528	435	219	......	......
25	Steam....	1.00	4.03	11.81	15.84	6.80	33	1 510	730	730	0.88	0.96
26	Electric.	1.94	4.31	0.66	4.97	9.49	30	1 556	647	333	......	......
27	Steam....	0.40	2.40	8.95	11.35	2.28	8	407	133	332	0.40	2.32
28	Electric.	3.26	6.73	1.08	7.81	18.25	60	3 309	1 637	503	......	......
Total	Steam....	25.21	59.64	92.36	152.00	150.51	734	35 581	12 617	......	18.96	......
Total	Electric..	28.16	67.15	33.54	100.69	155.75	743	27 783	12 800	......	......	......
Average.	Steam....	1.80	4.26	6.70	11.06	10.91	55	2 580	916	501	1.27	1.19
Average.	Electric..	2.01	4.79	2.40	7.19	11.13	53	1 980	914	445	......	......

HARLEM DIVISION, N. Y. C. & H. R. R. R. RUNS MADE BETWEEN OCTOBER 4TH AND OCTOBER 18TH, 1907.

SWITCHING, GRAND CENTRAL, WAKEFIELD AND NORTH WHITE PLAINS YARDS. — HAULING—STEAM, WAKEFIELD AND NORTH WHITE PLAINS—ELECTRIC, G. C. S. AND WAKEFIELD. — TOTAL FOR BOTH CLASSES OF SERVICE.

	Kind of Power.	Busy.	Waiting.	Dead.	Total.	Miles run.	Cars handled.	Ton-miles. Total.	Cars only.	Per busy hour.	Coal. Tons.	Pounds, ton-miles.	Water. Gallons.
Total	Steam....	14.12	71.87	98.36	170.23	77.68	16	13 843	249	......	14.98	......	24 000
Total	Electric..	11.31	46.57	40.43	87.00	81.96	11	7 920	137	......	......	......	......
Average.	Steam....	1.01	5.14	7.06	12.17	5.55	1	990	20	20	1.07	2.42	1 720
Average.	Electric..	0.80	3.33	2.89	6.22	5.83	1	570	10	13	......	......	......

Items.	Hauling.	Total.
Pounds coal per ton-mile of cars.........	1.32	1.41
Gallons water per ton-mile of cars.......	0.95	1.01
Watt-hours per ton-mile of cars..........	52.3	85.3

Tons of coal are based on 2 240 lb.
Ton-miles are based on 2 000 lb.

However, on the same wage basis for 1907 as for 1906, the month of August, 1907, showed a decrease in cost of terminal locomotive and yard operation of nearly $3 000, although the number of cars in and out increased from 64 984 to 68 519. In other words, the cost of operation decreased 9% while the work done increased 5½%, which is equivalent to a net saving of 13½ per cent.

Increased Revenue.—As to the fourth expectation—increased revenue from a larger volume of business—no definite conclusions can be reached until the extension of electrical service and the completion of the various other improvements afford an opportunity for increase in frequency and speed of train service; for the production of revenue from various sources at the terminal; and for the expansion of business that is sure to follow the enlargement of the facilities of the company throughout the suburban zone, not only as regards the local service, but in an even larger degree from long-haul freight and passenger traffic.

Summary of Results.—To summarize, the observations thus far made demonstrate that this pioneer electric installation in heavy-traction trunk-line work in the United States has fully accomplished the purposes that prompted its adoption, namely:

(1).—Abolition of nuisances incident to the steam locomotive; and

(2).—Increased capacity of the Grand Central Terminal,

a full year in advance of the date fixed by law; and in addition:

(3).—The promise, with the completion of the changes, of a saving, in cost of operation, of from 12 to 27%, after providing for increased capital charges for electrification; and

(4).—The outlook of a large future growth of remunerative traffic, and other sources of revenue attendant on the use of electricity, much more than sufficient to provide for the increased capital charges for the other improvements.

Several years will be consumed in the gradual rounding out of the work as a whole; but it is gratifying to have this early indication of the success of the undertaking from both the engineering and financial standpoints.

Other Operating Conclusions.—Apart from these results, it is interesting to note the conclusions, suited to this particular problem, that may be drawn from a study of the various observations.

Equipment designed for the electric system over which it is to operate offers economies so superior as to overshadow any other advantages that may be claimed for a kind of equipment that can be operated over several systems.

In switching service, the economy of electric traction lies in savings for supplies, and in lower unit fixed charges and repairs due to less lost time for repairs and care.

In slow-speed hauling, the advantage lies in the lower unit fixed charges and repairs of the electric locomotive, due to its ability to do more work while busy, and to less lost time for repairs and care.

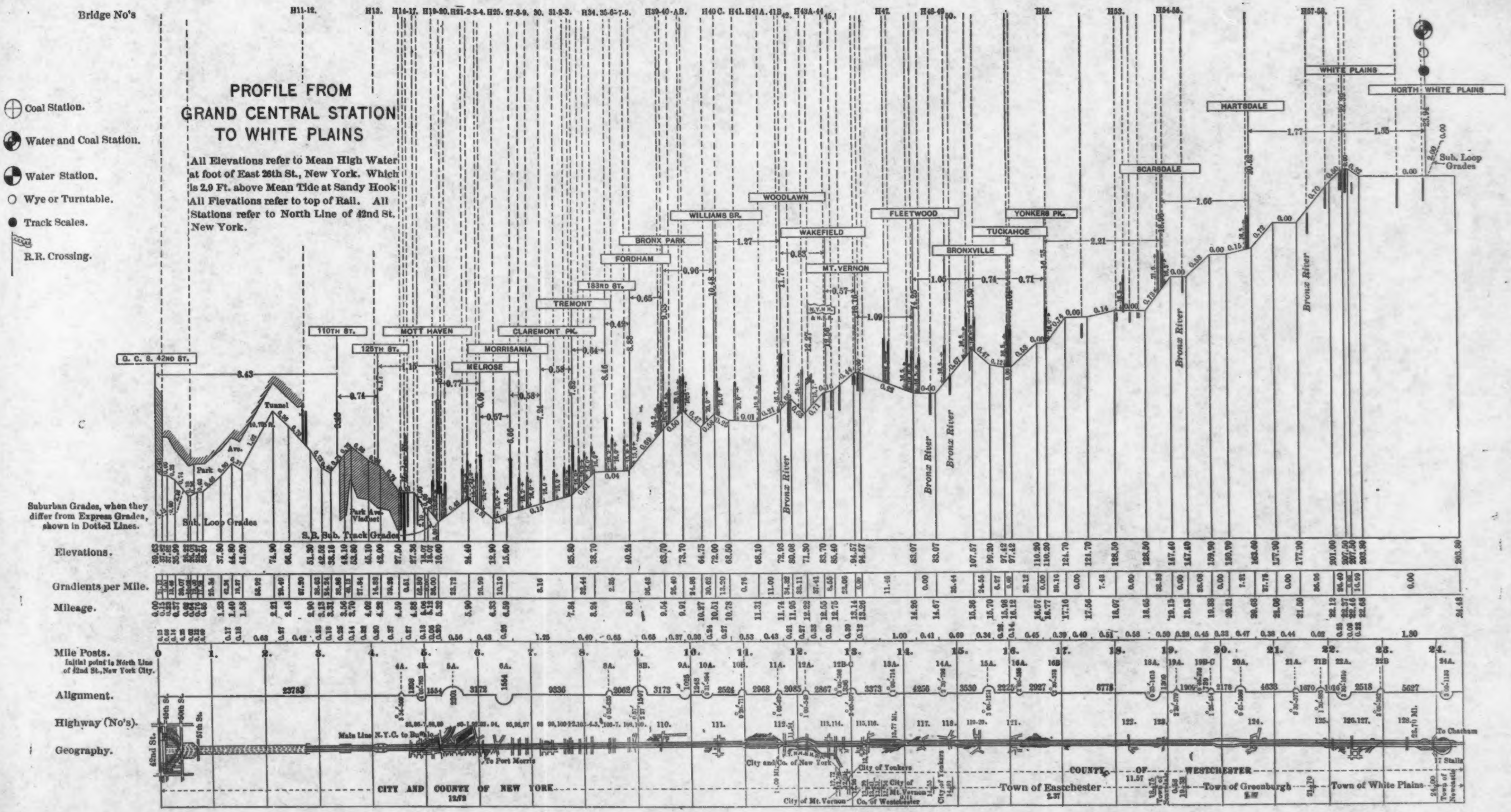
High-speed road service shows advantages for electric traction in all three items: supplies, wages, and fixed charges and repairs. The small 18% increase in current consumption for the greater speed of road service, as compared with hauling service, is in marked contrast to the 165% increase in coal consumption for steam traction.

Opportunities for large economies lie in the thorough training of motormen in the manipulation of their controllers, a very simple problem as compared with the difficulties of teaching both the enginemen and firemen on steam locomotives to perform their duties so as to result in fuel economy.

Maintenance of Track and Structures.—It is yet too early to express in dollars the comparative effect of steam and electric traction on the cost of maintaining and renewing tracks and structures. Repeated systematic inquiries of all foremen in charge of electric zone track maintenance, and of the motormen operating electrical equipment, have brought out the practically unanimous opinion that the effect of electric locomotives, apart from slightly greater wear on switches, does not differ from steam motive power, on either line or surface of tracks, but that the former has better riding qualities. The superiority of electric traction is manifest, of course, in the cessation of costly corrosive action of locomotive gas on metallic structures, and the freedom from cinders which, with the steam locomotive, cause heavy maintenance costs for cleaning rock ballast, and pointing brick tunnel arches.

Personnel of Engineering Department.—This paper would be in-

PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1079.
WILGUS ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.



complete without an expression by the writer, first as Chief Engineer and later as Vice-President of the Company, of his deep appreciation of the enthusiastic co-operation, and the able and earnest assistance of those associated with him in bringing this work to a successful issue.

The general principles and policies on electrical matters were determined by an Electric Traction Commission, the members of which, in addition to the writer as Chairman, were Bion J. Arnold, M. Am. Soc. C. E.; Frank J. Sprague, M. Am. Soc. C. E.; George Gibbs, M. Am. Soc. C. E.; and the General Superintendent of Motive Power, Rolling Stock and Machinery, New York Central Lines, at first Arthur M. Waitt, M. Am. Soc. C. E., later succeeded by Mr. John F. Deems. Mr. Edwin B. Katte, Electrical Engineer of the company, acted as Secretary.

The other improvements, including the preparation of plans and specifications, and the execution of all work, including electrification, by contract and by company forces, were under several corps, the heads of which, for the purpose of co-ordinating their efforts, formed the Construction Committee. Among the members of this Committee, including the writer as Chairman, were Mr. Charles A. Reed, Executive of the Associated Architects for the Electric Zone, Messrs. Reed and Stem, and Warren and Wetmore; Mr. W. H. Knowlton, Principal Assistant Engineer; Mr. E. B. Katte, Electrical Engineer; George A. Harwood, M. Am. Soc. C. E., Terminal Engineer; Mr. Azel Ames, Signal Engineer; Mr. George A. Berry, Engineer of Company Forces; Mr. Henry A. Stahl, Office Engineer; Mr. Victor Spangberg, Designing Engineer; and Mr. J. L. Holst, Bridge Engineer. Associated with this Committee were also Mr. Ira A. McCormick, General Superintendent of the Electric Zone, and A. T. Hardin, Assoc. M. Am. Soc. C. E., Assistant General Manager, as representatives of the operating department.

To the many contractors, who with few exceptions performed their work with skill and fidelity, appreciative acknowledgments are also extended.

DISCUSSION.

Mr. Henderson. G. R. HENDERSON, Esq.—This paper gives such a very interesting description of an important engineering accomplishment, that the speaker could add nothing to its value by praise, nor does he see any reason to criticise it.

The facts given are extremely valuable because they are the first statement of actual electric operation on a steam railroad, and give a fair and just comparison between steam and electric traction in trunk-line service.

Much has been heard in the last few years of the great economy that would result from electrification, but all figures were usually mere estimates, with a strongly suspected leaning toward the side in which their author was most interested. Such data were obviously received with suspicion, and perhaps incredulity, but Mr. Wilgus has now given figures which are not open to such questioning, and it has seemed to the speaker that some further analysis and study of these statements would be interesting and instructive.

In the following summary, the fundamental items were obtained from the pages noted, and the deductions were based on these figures.

ANALYSIS OF DATA.

Nominal generator capacity of each electric power-station, 20 000 kw. (page 78).

(Each power-house was designed to be able to carry full load, and two houses were established as safeguards against failures when inaugurating electric service—later, they are expected to be operated to full capacity) (page 77).

Average expected output for train propulsion, $\frac{107\,000\,000}{365 \times 24} = 12\,000$ kw. (page 79).

Maximum expected output for train propulsion = 24 000 kw.

Allowing for transmission losses, $\frac{24\,000}{9\,800} = 2.44$ h.p. per ton of train,

and as the horse-power $= \frac{\text{Tractive force} \times \text{Velocity}}{375} = \frac{(12 + 10) \times 40}{375}$ = 2.4 h.p. per ton of train at 40 miles per hour up a ½% grade, or equivalent acceleration.

The maximum number of trains in motion = 38, and $\frac{9\,800}{38} = 258$ tons, average weight of trains.

The maximum movement is double the average.

The nominal capacity of both power-stations is 1.7 times the maximum load, or 3.3 times the average load. One station, only, is 0.85 times the maximum load, or 1.7 times the average load.

Mr. Henderson.

Road service costs per 1 000 car ton-miles (page 102).

	Steam.	Electric.
Supplies	$2.03	$1.37
Wages	0.28	0.31
Interest, depreciation and repairs to locomotives	0.46	0.34
	$2.77	$2.02

The item, "Electric Supplies," is composed of operating expenses and fixed charges, and may be analyzed thus:

52.3 kw-hr. at $0.0109 = $0.58 for operation.
52.3 " " 0.0151 = 0.79 " fixed charges.

Total, 52.3 " " $0.026 = $1.37 (page 99).

The difference in cost between steam and electric traction in road service is $2.77 — 2.02 = $0.75 per 1 000 car ton-miles (page 102), and the fixed charges on the power plant and the transmission system are $0.79 per 1 000 car ton-miles, or about the same as the saving, so that if the train movement were but one-half the assumed amount (averaging 6 000 h.p. at the rails, or 6 000 kw. at the station), the cost for electric service would be slightly higher than for steam, or $2.81, as against $2.77 per 1 000 car ton-miles. This gives a ratio of power installation to average train requirements of $\frac{40\,000}{6\,000} = 6.6$, for equal costs of steam and electric operation, under the conditions and unit costs given by Mr. Wilgus.

In such a case, no saving in operating costs would result, and the contingent expenses of station and terminal alterations would usually be wholly unwarranted. The method of operating freight trains in "fleets" (especially on single-track roads, and with traffic in live stock) would require power-stations and transmission systems out of all proportion to the average load. It must also be borne in mind that smaller installations would cost more per unit for construction and operation, thus reducing the ratio or increasing the "load factor" for equal costs.

Much has been heard about "density of traffic" being the all-important consideration in determining whether or not electrification would be a financial success, but it seems to the speaker that uniformity of movement is equally important. Two properties, having the same average density of traffic, might have such different "load factors" that the fixed charges on investment would be two or three times as much in one case as in the other. Power-stations and distributing systems must be constructed for the maximum requirements, and if this maximum is greatly in excess of the average, both the operating expenses

Mr. Henderson. and the interest charges per kilowatt-hour will be increased. Thus, as has been seen, an average movement on the New York Central only one-half of that expected, would throw the monetary advantage to steam traction.

Besides, the enormous contingent costs (once stated by Mr. Wilgus to be about three times as great as the actual expense of electrification) could only be considered rational when the electric service would attract a great increase in traffic, and there are many cases where this would be a very doubtful proposition.

It is found, then, that not only density of traffic, but uniformity of movement should be given full consideration when investigating proposed electrification, and that, while the data derived from existing cases will be of assistance in prognosticating the results of a new venture, it is not the less imperative to discuss fully the proposition with its own environment, as no two cases will admit of the same arguments, and, if the investigation is made candidly and fairly, the cause of electric traction will be advanced more surely and successfully than by exaggerated statements impossible of practical fulfillment.

Mr. Gibbs. GEORGE GIBBS, M. AM. SOC. C. E.—Mr. Wilgus' paper is an admirably clear and concise description of the highly important New York Central electrification. He has accomplished the difficult performance of conveying in the space of a few pages of text and illustration a description of an extensive work, without sacrificing clearness, or unduly emphasizing some particular features to the exclusion of others.

The speaker presumes that the portion of the paper headed "Results" is the one which will most interest the average railroad man, and these few remarks will be confined to this portion. The speaker has had some three years' experience with two other important steam railroad electrifications, namely, the Long Island Railroad, with about 100 miles, and the West Jersey and Seashore Division of the Pennsylvania Railroad, with about 150 miles of electrified track, and he is still delaying the summing up of conclusions as to comparative steam and electric operating costs until the new method of traction has been in service long enough to determine with fair constancy its operating characteristics. Unfortunately for the purpose of making comparisons, all electrifications made to date (except those of urban traction), have been planned for anticipated extensions, and for a density of traffic far beyond that reached initially; the electrical results, therefore, do not show as favorably compared with the steam as it is hoped will soon be the case. It is noticed that Mr. Wilgus has also been obliged to forecast the future, in calculating savings, and, therefore, it would appear that it is still too early to say that we have accomplished our hopes in the samples of steam railroad electrification made to date, although each year brings the desired result nearer.

The author's remarks are confined specifically to New York Central conditions at the New York Terminal, and, therefore, one must be careful to apply his figures to this terminal only or to others having similar conditions. There are only a few very large terminals in the United States where train movement, both through and local, is extremely heavy and congestion is great. In such terminals the enormous expense for reconstruction and for electric equipment may be warranted, because of resulting increase of capacity, or because of public needs in tunnel operation, but, in the smaller cities, or where the service is light, the showing for reconstruction and electric operation would be extremely poor, and the expense altogether disproportionate to the results. Mr. Gibbs.

Mr. Wilgus, on page 103, sums up the advantages of electric operation, and gives, among others, "the promise, with the completion of the changes, of a saving, in cost of operation, of from 12 to 27%, after providing for increased capital charges for electrification." The speaker, understanding that this is not a present condition, but rather an anticipated future one, has endeavored to check this prediction with the results of operation on the roads with which he is connected.

The first important factor is the cost of electric current, which the author calculates at 2.6 cents per kw-hr. delivered at the contact shoe, and states that this includes all operating and maintenance costs, interest on investment required to produce and deliver the current, depreciation, taxes, insurance, and transmission-line losses. The speaker finds that this figure is lower than the corresponding one on the roads previously mentioned, yet he believes that the author states it very fairly for the ultimate result, based on a very heavy traffic and a large yearly output of current. This output is estimated at 121 000 000 kw-hr. yearly, or from five to six times as much as the present output of either of the roads with which the speaker is connected—and for the present New York Central, probably.

In Table 1 (Cost of Locomotive Service), interest and depreciation show, of course, much higher figures for the electric than for the steam locomotive, on account of the higher first cost of the electric machines. The author puts repairs for electric locomotives at two-fifths of those for steam. This figure appears to be based on insufficient practical experience with these machines. The speaker is led to hope, and even to expect, that the cost of repairs will be less on electric than on steam locomotives, but, how much less, he is not prepared to say. Test results he does not consider conclusive. It should be remembered that electric-locomotive design is in an undeveloped state, and constant changes in details, and even of type, are to be expected for some time; the cost of such changes must be charged to operation, and it would not be surprising to find that for the next few years electric locomotive repair costs will be disappointingly large.

Mr. Gibbs. The cost of handling and inspection (Table 1), including fixed charges, is placed at six times as much for steam as for electric locomotives. It would appear that in the case of terminal-zone operation, where steam and electric locomotives are interchanged, it will be necessary to maintain a steam-engine round-house as well as an electric locomotive house, with their respective organizations, also a yard for promptly interchanging locomotives on trains. It would seem, therefore, that Mr. Wilgus has allowed too much difference in favor of steam locomotives in this item.

Some of the other savings stated for electric operation are predicated on the greater car- or ton-mileage obtainable from electric locomotives. This may work out to be true, and doubtless will in some cases, but it is not always easy or economical to get the theoretically large electric locomotive or motor-car mileage out of such equipment on a steam road; it means that there must be trains on hand to haul, or that the service methods must be changed to short and frequent trains. This increases the cost for train crews, occupies track and terminal facilities, and, generally, is uneconomical, unless the local conditions permit, and the travel demands it. In practical operation, the speaker has noticed a tendency for reversion to steam-railway methods of heavy train units, where electric operation has been inaugurated on the opposite plan.

It would appear that Mr. Wilgus has calculated his costs, for interest and depreciation, upon the cost of the electric equipment only. It is a question whether or not this is a correct basis; in the speaker's experience, the electrification of a steam railroad involves numerous changes in the existing physical property of the railroad, and the cost of making these collateral changes will often equal the cost of the electrical equipment itself, and will seldom, if ever, fall below 50% of such cost. The New York Central electrification was undertaken for two purposes, and has resulted in the entire re-construction of the terminal proper, the yards, and the approaches. Each one of the objects sought, namely, the elimination of smoke and the reduction of congestion, depends upon the other. Thus, relief from congestion is expected to be accomplished by electric operation and by a re-design of the terminal, and part, therefore, of the cost of the electrification might be charged against the increased capacity of the terminal. On the other hand, to electrify properly required large collateral expenditures in the way of physical changes in the terminal, yard, approaches, tracks, and stations, so that some part of this expense should be charged against the main result, namely, the avoidance of smoke. If, therefore, any considerable part of the terminal reconstruction is charged in this way, it might alter very materially the figures Mr. Wilgus gives for possible savings.

These comments are not made in any critical spirit, but it seems to the speaker to be important that railroad men should appreciate the fact that electric traction has other aspects than that of cost. The New York Central Company has done a great work, to serve the public comfort and convenience, and to increase the safety in tunnel operation. It seems to be too much to expect that the change will be a money-making one, and it is to be hoped, therefore, that its progressiveness will at least earn an indirect reward in the approbation of the public. Mr. Gibbs.

ARTHUR M. WAITT, M. AM. SOC. C. E. (by letter).—This Society should feel a just pride in being able to present to the engineering world the first comprehensive technical record of the complete change of operating power—from steam to electricity—on the most important and busiest section of a large steam-railroad system; and the Society surely owes to Mr. Wilgus a debt of gratitude for presenting this most important and valuable paper for discussion. Mr. Waitt.

In these days, when there seems to be such a wide difference of opinion among enthusiastic electrical engineers, whose experience has been principally with light interurban passenger transportation, it is indeed refreshing and reassuring to have, in very clear and complete shape, not an unsubstantiated statement of what "probably may" be accomplished by this or that system of electrification, but rather a statement, fully proven in practice, of the accomplished results of a system which has proved successful to a degree which shows the exceptional ability and good judgment of those who conceived and executed the plans.

Having had many years of experience, and being familiar with the intricate problems presented to steam railroads at busy terminals, the writer can state without hesitation that the successful accomplishment of the change from steam to electric power at the New York terminal of the New York Central and Hudson River Railroad—the most congested railroad terminal in the world—with but little derangement of the train service, is but little short of a miracle; and, up to the present time, it may be properly rated as one of the most difficult and complicated engineering problems which has been undertaken and successfully carried through.

So carefully was the electrification problem studied by Mr. Wilgus and his associates, and so wisely were their decisions made, that the work went forward steadily and successfully in every stage, so that, after the first electrically-propelled trains were started, steady progress was made, without hitches or falling back, until, for the greater part of a year, every New York Central train leaving the Grand Central Station has been handled successfully and satisfactorily by electricity, with no instance where it became necessary to return to the use of the steam locomotive.

Mr. Waitt. This accomplishment speaks volumes for the wisdom of the officers in selecting their system of electrification. Especially is this manifest when it is seen that a closely associated railroad, adopting another system, has had failure after failure, so that it became necessary in several instances to abandon temporarily the use of electricity and return to the steam locomotive in order to run its trains. On this other railroad, instead of being able to make steady and continuous progress in the number and character of trains handled by electric power, the reverse has been the case, and there has been a decrease in the number of trains handled electrically; also, for some reason not yet made public, this road has not been able to handle heavy through trains satisfactorily by the system and apparatus which it has adopted.

Since the decision by the New York Central officers as to the system of electrification to be used, the writer has made an observation trip in Europe, and has been fully convinced that Mr. Wilgus and his associates have selected wisely, and have thereby avoided the various experimental forms of electrification, and many of the sources of difficulty, which have given no end of trouble in European continental practice. Conditions in Europe, as far as observed by the writer, are much simplier in most respects than those which prevail at the New York terminus of the New York Central Lines. The types of rolling stock, the absence of frequent overhead structures, the slower speed, and the lighter and less frequent traffic permit of experiments which are in many instances far from satisfactory, and would not be quietly tolerated by the American public.

One of the most important innovations in the installation on the New York Central Railroad is the use of the "inverted" or underneath-contact third-rail, which not only gives an absolutely safe working conductor, but has demonstrated its freedom from inefficiency in snow or sleet storms. The writer has seen in his European observations a line, equipped with the top-contact third-rail, almost completely at a standstill on account of a slight coating of sleet on the top of the third-rail, which prevented the contact shoes from collecting sufficient current to keep the trains in motion, notwithstanding the fact that men, equipped with brooms, were located on every half-mile section of road to keep the rails clear.

European practice has been largely of negative value in determining the best practice for America, by indicating what to avoid, rather than what to adopt. American engineers have set the pace, and it is safe to say that the most successful systems in use in Europe to-day are those modeled after American designs and principles.

Mr. Lewis. Nelson P. Lewis, M. Am. Soc. C. E.—The speaker can contribute nothing to the interesting discussion of the relative merits of alternating and direct-current equipment, or of the third-rail and the overhead conductor, but he wishes to do a simple act of justice in calling

attention to the attitude of the author of the paper and the corporation which he represented during the negotiations leading up to the adoption of the plans now being carried out. Mr. Lewis.

These negotiations and plans had their origin in the movement to substitute electricity for steam in the Park Avenue Tunnel, and to increase the capacity of the Grand Central Yard. Then, in order to restore to public use the eleven streets from Forty-fifth to Fifty-fifth Streets, inclusive, the continuity of which was broken by this terminal yard, the Company agreed to depress the entire yard many feet into the solid rock of Manhattan Island at an enormous increase of cost.

When the terminal station was under consideration it was first proposed to make it a revenue-producing structure, containing not only offices for the use of the company, but a hotel, and possibly a theatre and stores, with offices for rent. As the plans were being developed it was concluded that an improvement of this kind demanded the erection of a monumental building devoted exclusively to railroad purposes, and the plans were accordingly modified with this in view. It was then believed that such a building should have a proper setting, and the company, of its own volition, proposed to place it some 40 ft. north of the northerly line of Forty-second Street and some 70 ft. east of the easterly line of Vanderbilt Avenue, and to throw out to the use of the public this area, equivalent to about twenty-five city lots each 25 by 100 ft. The market value of the land thus surrendered is approximately $1 250 000. This was not done simply to secure accommodations for its own patrons, as they already had right of access over a public street, but the company has also acquired from the city sub-surface rights beneath Vanderbilt Avenue to be used for the accommodation of cabs and carriages, and for these rights it is paying the city an annual rental based upon the value of the adjoining property.

The speaker takes pleasure in emphasizing to the Society the broad and liberal spirit with which Mr. Wilgus and his associates treated the important and difficult problem which was presented to them; they considered it not only as a railway problem, but as one which had an important relation to the dignity and beauty of the great city in which their terminal is located.

H. M. Brinckerhoff, Esq.—This paper is particularly interesting to the speaker, for a number of reasons. One of these—shared in by all who have been engaged in electric railway work—is that this is the first authoritative expression from an official of a large steam railway system strongly endorsing electric traction as applied to steam railways. Mr. Brinckerhoff.

Having been actively engaged for the past eighteen years in electric railway work, and particularly in the original design and development of the third-rail system applied to the Intramural Railway at

Mr. Brinckerhoff.

the World's Fair, in 1892-93, and later having charge of the equipment and management of the Metropolitan West Side Elevated, in Chicago, this paper appeals to the speaker, because it marks an epoch in the history of electric traction.

When, some 25 years ago, it was proposed to supplant horses on street railways, and use electric motors, a great deal of doubt was expressed, even by well-informed people, as to the possible success of such a venture. When the street trolley system became an established and successful fact, and some talk arose as to its possible application to heavier types of traction systems, those familiar with railroad work were fond of saying:

"Electricity is well enough as a substitute for a horse; it does very nicely for street cars, where everything is so light and the service conditions are not exacting, but, for a real railway, it is obviously unsuitable."

When, in 1892, it was proposed to operate electrically an elevated railway on the World's Fair Grounds at Chicago, and, further, it was proposed to use a steel rail as a conductor and a cast-iron shoe with gravity contact as the collecting device, the same authorities good-naturedly admitted that:

"This was well enough as an experiment, or an advertisement, or an exhibition, and we might pull through, as it was only for use during the summer season, but that, as a development applicable to steam-railway work, it was of course obviously unsuitable."

When, a year or two later, this same system was successfully applied to the Metropolitan West Side Elevated Railroad, in Chicago, followed by the abandonment of the steam locomotives on all the elevated roads in that city, and later supplanted steam on the Brooklyn and Manhattan Elevated Systems, and was given first choice on the New York Subway and on the Boston Elevated, as well as successfully operating long interurban lines in the Middle West, the doubters again retreated behind a further line of defense by saying that:

"This was simply a modified form of horse-car service, and that, obviously, while suited to lighter forms of traction, it was not applicable to steam railways."

Now comes a further and vital step in the forward progress of this art. The representative of a great steam railroad system comes forward and publicly announces that, not only does the electric apparatus substituted upon this company's great terminal succeed in actually performing the work previously thought possible only with steam locomotives, but that it actually is able to accomplish 30% more with a given terminal capacity, and at an actual saving of 13½% in operating expenses.

This admission cannot but be hailed by all who have borne the burden and heat of the day, while struggling with the harrowing de-

tails of this work, as a great step toward a broader and more general appreciation of the possibilities of their favorite power, and a signal indorsement of the claims and arguments put forward in its favor during the past few years. Having announced these important facts, Mr. Wilgus has contributed a text upon which may be expended a great deal of argument. Mr. Brinckerhoff.

The speaker would like to bring forward, however, two points represented specially by his own experience.

This paper confirms by actual demonstration a number of the claims which have been put forward from time to time in favor of electrifying portions, at least, of the steam railroad systems of the country.

The fact that at least one-third additional capacity has been obtained for a given number of tracks in the old station, by the simple change to electrical operation, has an important bearing upon terminal capacity, where space may not be so restricted as to warrant the great expense of a two-deck construction. That it was possible to make this or even greater increases in capacity of tracks, yards, and crossings by electric operation over steam, has been admitted for some time, even by steam railway men, in the case of the lighter forms of traction work, such as the elevated railway and kindred services. Now, having proved by actual experience the relative increase in capacity of electric over steam operation in the heaviest kind of terminal work, at the Grand Central Station, it certainly seems reasonable to follow one or two other lines of comparison in this regard.

Take, for instance, the question of operating costs. Mr. Wilgus has shown, as the result of his tests and deductions from operating statistics, that there is an actual decrease or saving in electric over steam locomotive operation of 9%, and an increase of work done of 5½%, being equivalent to a net saving of 13½ per cent. The question at once arises: Has the period under consideration, and from which the details of operating costs, repairs, maintenance, etc., on the locomotives, been sufficiently long to be a conclusive demonstration?

As an answer to this query Table 4 shows for comparison the operating data for very considerable periods—from 3 to 12 years—of steam and electric operation on elevated railway systems. The costs, in these cases, as is customary in systems of this kind, are kept per car-mile, and include all operating expenses, excepting only taxes and insurance.

During the eleventh year of operation of the Metropolitan West Side Elevated, coal was 20% and wages 14% higher than with the period of steam mentioned. This latter figure the speaker can vouch for, as he was Manager of the road for six years, including this eleventh year, and these costs include current renewals of all kinds, curves, crossings, rails, ties, painting cars, and renewal bodily of large parts of the equipment.

Mr. Brinckerhoff.

TABLE 4.—COMPARISON BETWEEN STEAM AND ELECTRIC OPERATION ON ELEVATED RAILWAY SYSTEMS.

	Steam.	Electric.	Decrease.	Increase.
MANHATTAN ELEVATED, NEW YORK CITY. (Third year of operation.)				
Cost per car-mile	$0.1198	$0.095	20.4%	......
Speed, in miles per hour	10.1	15	......	48.5%
OAK PARK AND CHICAGO ELEVATED. (Eighth year of operation.)				
Cost per car mile	$0.1174	$0.107	8.2%	....
Speed, in miles per hour	12.5	15	.....	20%
SOUTH SIDE ELEVATED, CHICAGO. (Seventh year of operation.)				
Cost per car-mile	$0.106	$0.089	16%	......
Speed, in miles per hour	13.08	14.95	....	14.3%
METROPOLITAN WEST SIDE ELEVATED, CHICAGO. (Eleventh year of operation.)				
Cost per car-mile	$0.11*	$0.0931	15%	....
Speed, in miles per hour	12*	15.4	....	28%

* Assumed.

The fact has been clearly demonstrated that the operation of these elevated systems during a period of from 10 to 12 years has produced no maintenance charges which tend to bring the cost within 12 or 15% of the best similar steam operation; at the same time, schedule speeds 18% higher than the steam, and capacity at junction points, terminals, etc., from 30 to 40% greater are regularly maintained.

After a consideration of these facts, as proven during periods of operation which have developed full normal maintenance charges on these lighter forms of electric traction, it seems perfectly reasonable to assume that all the other points of similarity, having now been demonstrated on the heavier service, this continued low maintenance is reasonably to be expected. The apparatus in general, and even in much detail, is identical in principle, varying only in the dimensions of parts. The great simplicity of the driving mechanism throughout, and the absence of any reciprocating parts, applies equally in both cases.

The experience gained in one field, the speaker believes, can be applied with increased confidence in the other, now that Mr. Wilgus has given this authoritative statement of this successful demonstration of the initial period on a steam railway installation.

The author's reference to the expected increase in suburban traffic, due to the electrification when carried to Harmon and White Plains,

is a point which those familiar with electric railway developments cannot but look upon as practically assured. This question of electrification of suburban zones by steam railroads is controlled in most cases, of course, by local conditions. Where the right of way controlled by the railroad company is only sufficient to meet the demands of heavy through business, it is obvious that the company cannot afford to devote a set of tracks to purely local traffic at small fares, low enough to compete with possible street railway or interurban competition. Where, however, a railroad controls an ample right of way, or is itself operating a steam suburban service on separate tracks, the policy of continuing such unattractive and inadequate service, as is usually rendered in this manner, until a competing electric line has actually been placed in operation, seems to the speaker peculiarly short-sighted. Mr. Brinckerhoff.

If a territory will develop a traffic sufficient to pay well for the installation and operation of an independent electric line, which is put to the expense of purchasing a new right of way, why would it not pay the steam railroad to head off such competition by equipping its own suburban service with a type of apparatus which has proved such a dangerous rival in other cases? Would this not have a number of compensating advantages such as inducement to build and enlarge residence districts close to the steam railway line rather than scattering them out at a distance where the electric line becomes not only a passenger-carrier but often a freight and express handler for entire new communities?

These considerations, of course, are subject to local conditions, as before mentioned; but the speaker believes it is a fact that if many steam railway managers had the past five years to live over again, they would not allow themselves to be caught in the manner seen in a great number of instances, and to-day there would be more steam railroads under partial electric operation and fewer heavy electric interurban railroads in existence.

George B. Francis, M. Am. Soc. C. E.—In the discussion of this paper much has been said concerning the comparative merits of the application of electric current by the direct method through a third-rail, or by the alternating method through overhead contact, and little has been said about the difficulties or magnitude of the civil engineering work required in reconstructing the railroad generally, to make electric traction applicable, or the work required in new structures for the same purpose. Mr. Francis.

Someone has said that electrical and mechanical engineers are "dynamic" engineers, although such engineers, under the usual qualifications, are eligible to membership in the American Society of Civil Engineers. The "dynamic" engineers have certainly had full part in this discussion.

The Society as a whole, however, is largely made up of civil en-

Mr. Francis. gineers of another class, who are not "dynamic" engineers, but may be termed engineers of "statics" or "static" engineers, persons who design and build structures which it is not intended shall be moved or have the capacity of locomotion or moving action, but in which the static work is nevertheless very useful.

Civil engineers of this class are much interested in all that has been done, in their line, toward the application of electric traction, and they appreciate the work done on and along the electric zone by their professional brethren, who should be complimented for the results so well attained.

It has been the experience of those in charge of electrical traction work (where such traction power has been installed to take the place of steam power) that the expense of alterations and additions to the general "static" property has run very high, almost as high as the cost of the strictly necessary features for electrical operation *per se*.

This may not be so pronounced when long lines of steam railway are equipped electrically, but, up to the present time, the application, in most instances, has been made in the vicinity of large terminals or in congested districts, and the costs pertaining to matters extraneous to power and equipment have been generally under-rated.

The electrification of the New York Central, as described, has embraced such extraneous work as follows:

The purchase of a great many acres of very expensive land,
The widening of cuts and fills,
The removal of all grade crossings, aggregating forty-four, altogether,
Yard construction for changing from steam to electric power,
Shops for electrical repairs, and inspection sheds,
Bridges for restored city streets in the vicinity of the Terminal Station,
The reconstruction of portions of the Park Avenue Tunnel for track room,
The entire removal of the Grand Central Station, and its reconstruction, with two decks for trains,
The removal of the Mott Haven Station, and its reconstruction in a new location,
The creation of temporary facilities during reconstruction,
Additional track bridging,
The reconstruction of all signal systems,
Additional main tracks,
Changes in grades of tracks, which, together with new trackage has required the excavation of 3 000 000 cu. yd. of rock and earth (mostly rock)* at the terminal site,

*This material has been hauled to points miles away, and the work and haulage have been done without interference with daily traffic.

PLATE XXIII.
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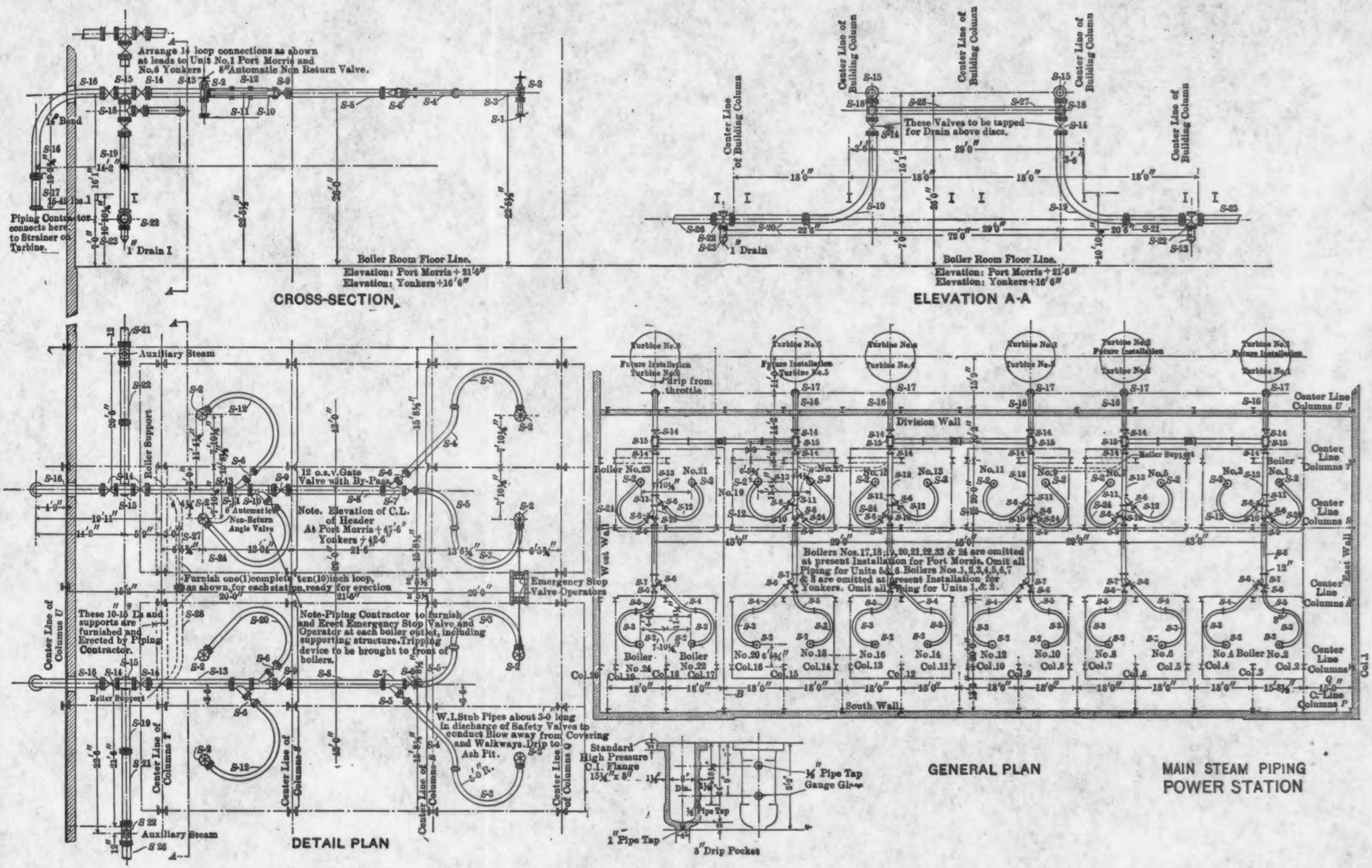
KATTE ON
ELECTRIFICATION OF SUBURBAN ZONE
OF N. Y. C. & H. R. R. R.



MAIN STEAM PIPING
POWER STATION

An entirely remodeled junction of tracks at Mott Haven, to avoid grade crossings of tracks, **Mr. Francis.**

About a dozen local stations have been entirely remodeled with subway and overhead bridges, in order that passengers need not go upon the tracks.

Many other important improvements have been schemed and carried out, and are fully described in the paper. This list is sufficient to emphasize the cause of extraneous cost.

Regarding the "static" engineering features required to produce the necessary parts for electric operation, the following may be mentioned:

Foundations for the power-house at Yonkers, and the house itself,

Foundations for the power-house at Port Morris, and the house itself,

Coal-handling plants,

Sub-station buildings and other minor buildings,

Underground conduits,

Repair shops and interchange terminals.

To design this work in the office, to lay it out on the ground, and to inspect and supervise it during construction, as well as to prepare all the plans for land purchases, with the accompanying deed and condemnation descriptions, has been the work of the "static" engineer, and it is to be hoped that, at the proper time, a complete paper, describing this kind of construction, together with a description of the new Grand Central Station and terminal, may be prepared and presented to the Society as supplementary to Mr. Wilgus' paper on "electrification."

EDWIN B. KATTE,* ESQ.—Mr. Wilgus has covered the subject so completely that it does not seem possible to add much of general interest; however, some of the details of the work already described will perhaps be of value to those interested in a further consideration of this electrical installation. **Mr. Katte.**

Storage Batteries.—Mr. Wilgus has explained that storage batteries were installed as an insurance against interruption to the train service; this value was strikingly illustrated a few months ago, when, during the most severe wind storm in this locality for many years, several telegraph poles, which were on a high bank above the aerial transmission lines, were blown down, and one pole, with its numerous telegraph wires, hung suspended on the 11 000-volt aerial transmission lines. The effect was to open instantly the circuit breakers in the power-station, and the safety devices in the sub-stations automatically

* Electrical Engineer, New York Central and Hudson River Railroad.

Mr. Katte. disconnected from the load every rotary converter on the system. The batteries were "floating" on the bus-bars, and immediately took up the train load, and there was no interruption to service. The load despatcher, knowing that the batteries would carry the load for a sufficient time, was able, in an orderly manner, to locate the cause of the trouble and then direct the various operators in charge of the several sub-stations how to start up and throw in their rotaries and pick up the load.

CURVES SHOWING DIVISION OF LOAD BETWEEN ROTARIES AND BATTERY.
TWO 1500 K.W. ROTARIES RUNNING.
BATTERY HAS 67 TYPE "R" PLATES, AND IS EQUIPPED WITH CARBON REGULATOR.
READINGS TAKEN AT INTERVALS OF 5 SECONDS.
SUBSTATION No. 2. AUG. 7, 1907.

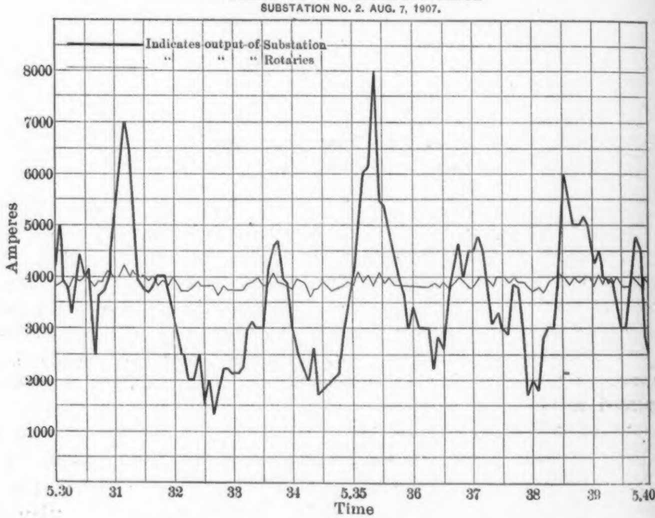


FIG. 8.

Another reason mentioned by Mr. Wilgus for installing storage batteries was to relieve the rotary converters and generators from the sudden fluctuation of load due to the starting, stopping and passing of heavy trains. That this has been effectually accomplished will be apparent from the diagram, Fig. 8, taken from actual readings at one of the sub-stations, the heavy line representing the output of the sub-station and the fine line indicating the load on the rotaries, the heavy fluctuations having been taken up by the battery. The readings were taken at 5-sec. intervals for a period of 10 min.

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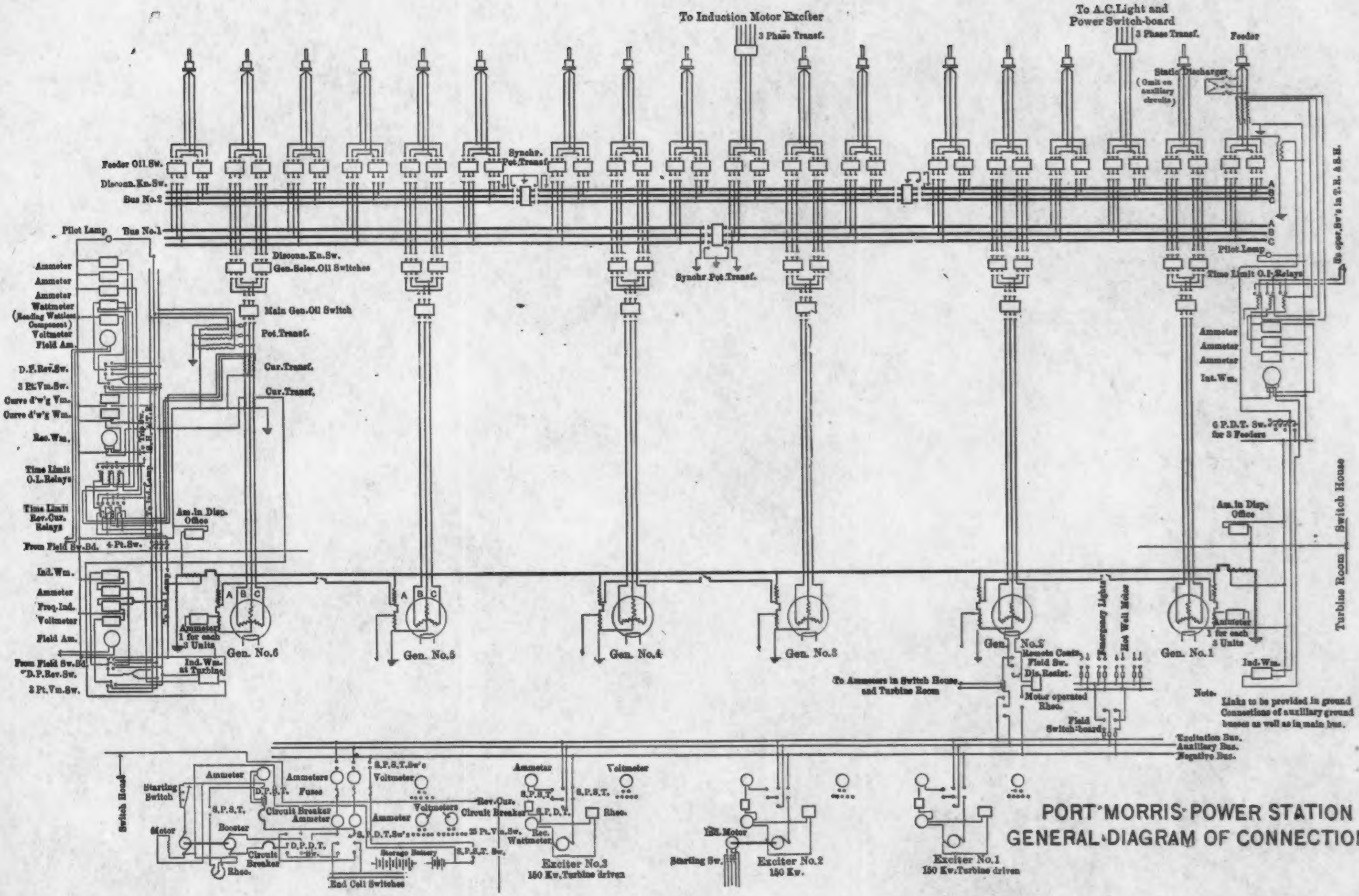

PORT MORRIS POWER STATION
GENERAL DIAGRAM OF CONNECTIONS.

Aerial Transmission Lines.—Mr. Wilgus has referred to the reliability of well-built aerial lines. Aside from the one instance just referred to, when telegraph poles fell on the line, there has never been an interruption in the high-tension aerial circuits; and it is of interest to note that the interruption mentioned only interfered with one circuit, and the amount of damage to that circuit was the mechanical breaking, by the swaying by the telegraph pole, of three out of seven strands of one conductor. It is the practice of this company to place the aerial transmission lines on the opposite side of the right of way from the telegraph poles, but, at the location above cited, it was necessary, for a space of three pole lengths, to place the telegraph and transmission line poles on the same side. Mr. Katte.

High-Tension Distribution System.—The 11 000-volt circuits have been laid out to afford the maximum flexibility with the minimum quantity of copper from the two main generating stations to the eight substations. From the diagram, Fig. 9, it will be noted that each substation, with the exception of the outlying stations, Nos. 6 and 8, has a direct circuit from the adjoining power-station, and in the case of Sub-stations Nos. 6 and 8, the supply is from direct feeders through Substations Nos. 5 and 7. Each substation is fed by two or more independent circuits, and in such a manner that either power-station can feed any sub-station. The more important sub-stations, namely, Nos. 1, 2, and 7, which supply current to the congested Harlem Division, are each fed by at least two circuits direct from the power-station. The duplicate circuits are entirely independent, being located on the two sides of the right of way, so that an accident to one circuit could not possibly affect the other.

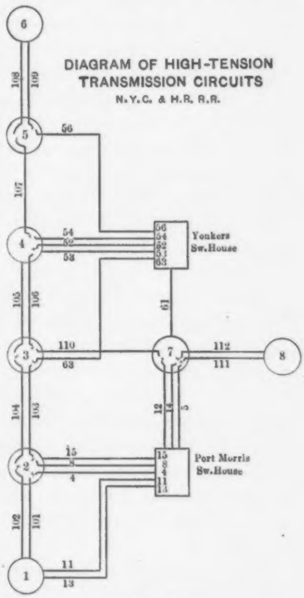

FIG. 9.

Power-Stations.—In addition to what Mr. Wilgus has said regarding the power-stations, a feature has been made of the sectionalizing of each unit. Each turbo-generator, with its condenser, auxiliary apparatus, boilers, feed pumps, etc., is a complete unit, and can be isolated from other units in the station in case of trouble. This is true of the piping arrangement as well as of the electrical connections. These

Mr. Katte. features are illustrated in the general piping plans, Plate XXIII, and the wiring diagram, Plate XXIV.

Switch-Houses.—The separate switch-houses are a distinctive feature of the Port Morris and Yonkers power-stations, and contain several novel features, although the idea of separate houses is not new, having been used at the Fisk Street Station, in Chicago. The two switch-houses, in generic principles, are the same, although the layout is different in each. Plates XXV and XXVI illustrate the salient features of the Port Morris switch-house. From the plans it will be noted that the low-tension or control wiring is kept separate from the high-tension circuits, thus eliminating all danger in making repairs to these circuits. By the use of barriers, every precaution has been taken to protect attendants from short-circuit flashes in the high-tension compartments. The elevation shows clearly the use to which the building has been put, the sub-basement being devoted entirely to conduits and manholes. The basement, which is on the same level as the main floor of the generating station, contains the bus-bar compartments, and the floor above is given up to the oil-switches, switch-boards and bench-boards. The second floor, which is on the level of the switch-board gallery, and is connected thereto by a bridge, contains the load despatcher's office, the exciter storage battery, the heating and ventilating system, locker-rooms, and storerooms.

Sub-Stations.—In the present electrification scheme there are eight sub-stations, located at the most economical load centers. Five of these stations are now complete, and four are in daily service. The general principles governing the design of all the sub-stations are the same, although the detailed arrangements vary somewhat in each particular case. The general features are shown in the typical drawing, Fig. 10. As in the design of the power-stations, the sectionalized-unit system has been carried out, as far as possible, and each rotary converter, with its transformers, switches, etc., is as independent as conditions will permit. The typical sub-station wiring diagram is shown on Plate XXVII.

Third-Rail.—The under-running third-rail has now successfully passed through the snow and sleet storms of three winters, the first year at the experimental track, west of Schenectady, and two years in actual service in New York where about 90 miles are in daily use. There has not been a single instance of a moment's delay due to sleet or ice on the third-rail, or snow interfering with collecting the current or the passing of trains. The adequacy of the third-rail protection is illustrated by the fact that not a single patron of the road has been injured on account of contact with the third-rail, and but few employees, while, in every instance of injury, it has been due to gross carelessness or negligence on the part of the employee injured.

Low-Tension Feeder System.—The direct current is in all cases fed to the third-rail through circuit breakers which are controlled from

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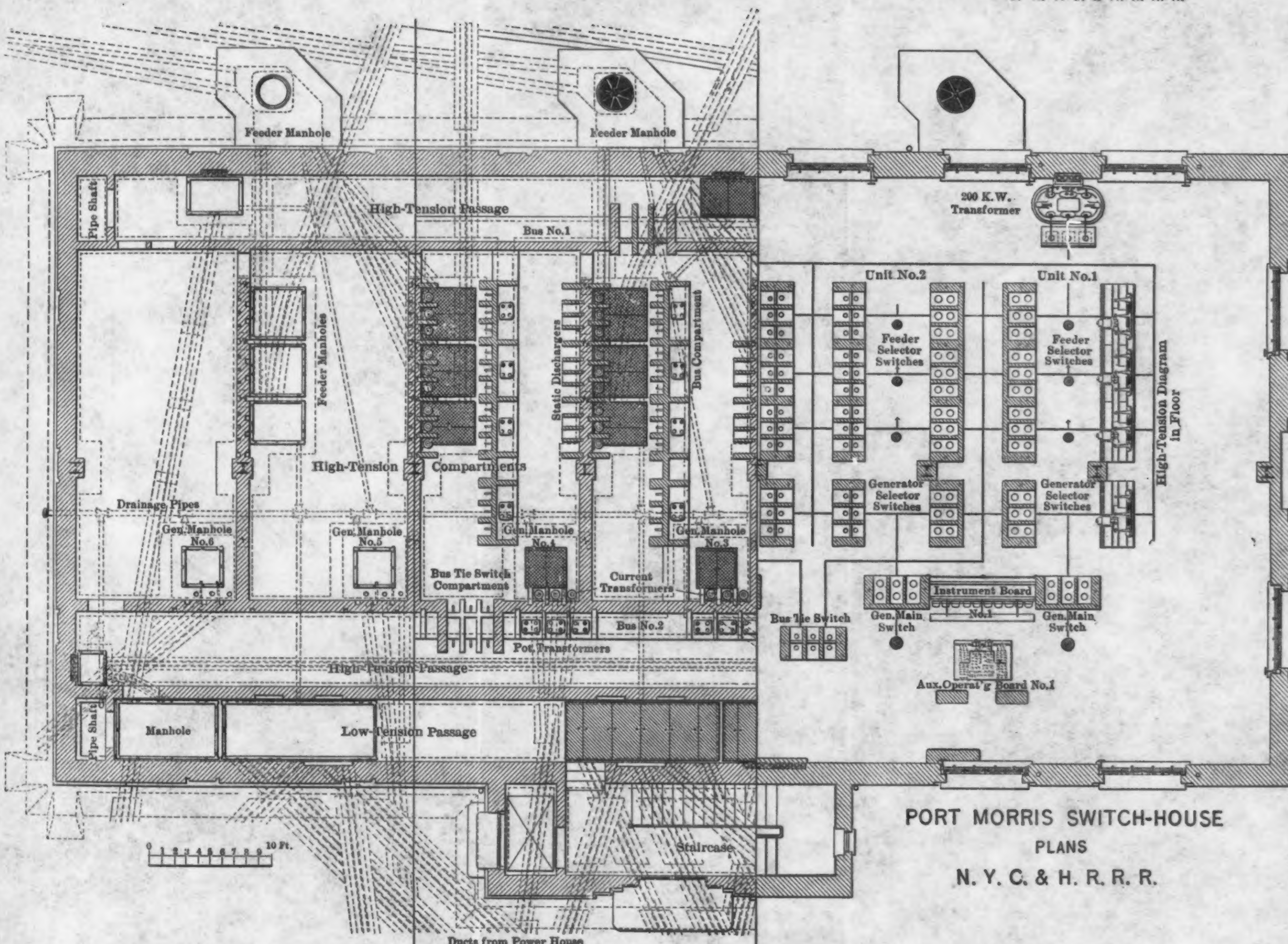

PLAN OVER BASEMENT

PORT MORRIS SWITCH-HOUSE
PLANS
N. Y. C. & H. R. R. R.

Mr. Katte.

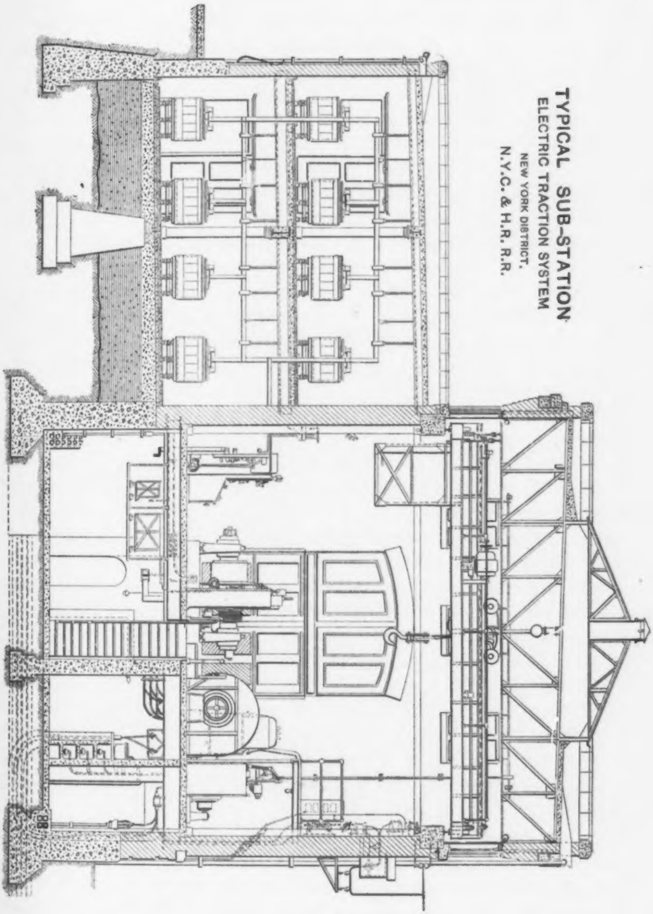

Fig. 10.

Mr. Katte. the nearest sub-station, and the entire third-rail system is paralleled by auxiliary direct-current cables; this permits cutting out all third-rails in a given section, and feeding around through the auxiliary cables to adjoining sections for the operation of trains on either side of the dead section. The diagram of positive feeders, Plate XXVIII, shows the various connections to the third-rails and the auxiliary cables. Opposite each sub-station, except the end ones, there is an isolated section of third-rail sufficiently long to take the longest multiple-unit trains. The purpose of this isolated section is to prevent a train bridging from a live rail to a dead third-rail section, on which section there may have been an accident, or on which men may be working, and further, to prevent bridging, two sections of third-rail in which, because of their being fed by different sub-stations, there might be a difference of potential and thus cause the blowing of the motor fuses in the train.

Precautionary Devices.—Among the devices to ensure safety to the system may be mentioned the indicating wire which has been woven into the protecting braiding of the direct-current cables along the Park Avenue Viaduct and through the Park Avenue Tunnel. The function of this wire is to trip the circuit breakers and notify the sub-station attendants by the ringing of a gong should a short circuit occur of sufficient severity to cause burning or injury to the cable, but not involving a quantity of current which would open the circuit breakers on an over-load. The need of this device was demonstrated last year on the Park Avenue Viaduct when a defective joint in a direct-current cable failed, and, because of the disobedience of an attendant to follow an order, all the switches to clear the short circuit were not promptly opened. The burning continued for some time because the quantity of current flowing was not of sufficient volume to open the over-load circuit breakers, which, necessarily, have to be set to carry large quantities of current for the regular handling of heavy trains.

A safety device installed in the Park Avenue Tunnel consists of a cord, running parallel to all tracks, and connecting with signal alarm boxes. If for any reason it becomes necessary to cut off current quickly from any third-rails, a pull on the safety cord will open the circuit breakers feeding that section of third-rail, and at the same time will notify the attendant at the sub-station that the third-rail has been cut out.

Mr. Murray. W. S. Murray, Esq.*—The thanks of the railway engineering fraternity are due to Mr. Wilgus for this paper.

The author's theme is broad, and in the accomplishment of the results severally cited, the speaker joins in the general assent regarding the marked successes which rightfully have been obtained in this stupendous work.

Only about half a page of this paper is devoted to conclusions with

* Electrical Engineer, New York, New Haven, and Hartford Railroad.

PLATE XXVI.
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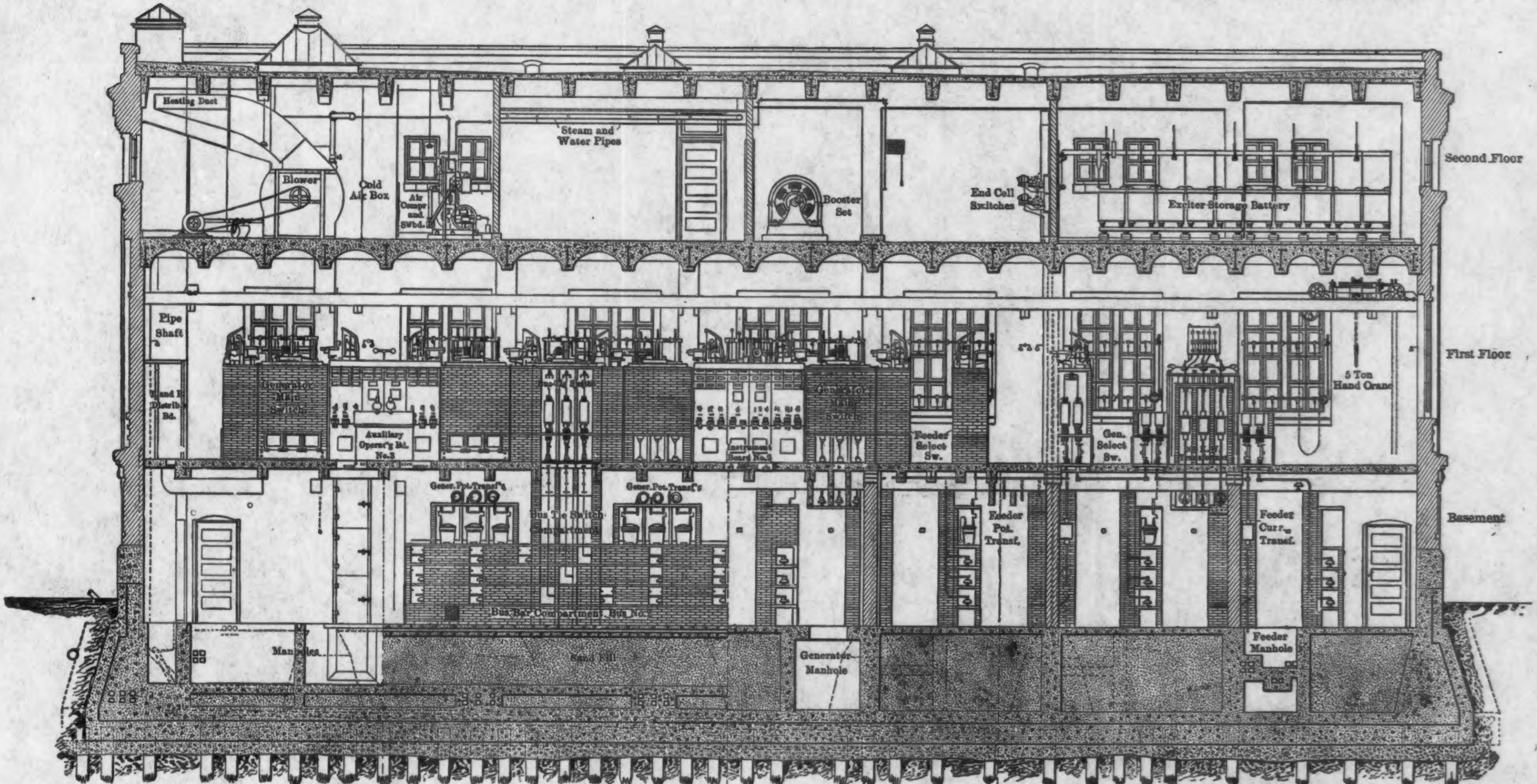

PORT MORRIS SWITCH-HOUSE

SECTION EAST-WEST

SECTIONS TAKEN AS FOLLOWS:

Basement: Low Tension and High Tension Passages and High Tension Compartments:
First Floor: Front of Aux. Op. Bd. No.3, Instr. Bd. No.2, 200 K.W. Transf. and through Oil Switches;
Second Floor: Heating and Ventilating Room, Hall, Booster- and Storage Battery Room.

Scale of Feet
0 1 2 3 4 5 6 7 8 9 10 Ft.

which the speaker is not in perfect agreement, and as there is such a small difference of opinion—in space at least—he will waive further comments. Mr. Murray.

In order not to be misunderstood in this matter, the speaker must state first that, given the New York Central zone of electrification, free for a decision as to the form to be adopted, he believes that a majority of the best informed electrical engineers of America would to-day cast their ballots in favor of single-phase electrification. If this statement causes surprise, it should be at once explained that it is not because the speaker has not been of this opinion ever since the New Haven road made its decision as to the form of electric traction it would adopt, but because he could find no reason to criticise what had already been accomplished. Mr. Wilgus makes the following statement:

"The wisdom of adhering to the type of equipment already chosen has been proven by recent comparative tests of locomotives of the two types under exactly the same conditions, which demonstrate that the one designed only for direct current consumes from 15 to 25% less current than the one intended for use on both systems. This will effect a saving to the company of at least $140 000 per annum."

This statement deserves careful analysis. Before discussing this item of economy, which, as stated later, becomes $300 000 per annum, a previous paragraph on the same page will be mentioned, in which are discussed the three principal reasons for adopting the direct-current system.

(1).—The insufficient practical development of the alternating-current system for a trunk-line problem requiring absolute reliability of service;

(2).—Restricted clearances, which forbade the use of overhead conductors;

(3).—The legal obstacles to the use of overhead trolley wires carrying high voltages within the limits of the City of New York.

(1).—Under "Reasons for Electrification of New York Central," Mr. Wilgus has stated that legislative action required the complete abandonment of the steam locomotive in Park Avenue, south of the Harlem River, within a period of five years, terminating July 1st, 1908. He also states that the change of motive power for schedule trains was completed on July 1st, 1907; thus, three or four years after the date of the decision to electrify, commercial trains were placed on schedule one year before the date required by the State of New York. A year previous to the decision of the New York Central to electrify, a paper was read, before the American Institute of Electrical Engineers, by Mr. Benjamin G. Lamme, Chief Engineer of the Westinghouse Electric and Manufacturing Company, descriptive of the Washington, Baltimore, and Annapolis single-phase railway, in

Mr. Murray. which diagrammatic power connections were shown, fundamentally in principle, as adopted to-day, together with practical data, in the form of traction, speed, power-factor, and efficiency curves. It seems to the speaker that the State of New York would probably have been satisfied if the full time allowed had been taken, and, instead of being one year ahead in operation, thought had been devoted to the single-phase proposition. If, at the expiration of this year, the author did not believe the art had been sufficiently advanced for its adoption, the speaker believes that his first reason would not have existed.

(2).—Restricted clearances, which forbade the use of the overhead conductors: There is a zone in which this matter has been handled. Where the author can point to one restricted clearance on his line, the speaker can point to five on the New Haven road, and does not doubt but that the overhead obstacle clearance on the New Haven will be found to be closer to the rails. The answer to this can be anticipated:—the overpowering argument—how would the Park Avenue Tunnel be electrified? How have the Simplon Tunnel, the Sarnia Tunnel and other tunnels been electrified? All these have restricted clearances, and yet there is nothing mystifying or difficult about the installation of overhead conductors under clearances of this character. In short, unless the clearances in parts of the New York Central electrification zone, other than those over which the New York, New Haven and Hartford trains operate, are of a character strangely different, the speaker would undertake to install the overhead type of construction.

(3).—The legal obstacles to the use of overhead trolley wires carrying high voltages within the limits of the City of New York: Even before that organization known as the Gas, Water, and Electric Light Commission was dissolved by the bill introduced and adopted under the present administration, Mr. Wilgus elected to erect his overhead conductors within the limits of the City of New York, and chose for their location, not points over the part of the railroad company's right of way, where their traffic is most dense, but at its edges, and there his transmission lines are carrying 11 000 volts. Why, therefore, if these voltages can be assimilated by the City of New York on the edges of the right of way, can they not be tolerated toward the center of that strip of land, which will place them farther from the public? The speaker ventures the assertion that now that the State of New York controls its public service corporations by a Public Utilities Commission, it will confirm Mr. Wilgus' action in placing these high-tension wires on the New York Central Company's right of way, even if he has elected to place them as near as possible to the public. In connection, also, with the question of high voltage, this discussion would not be complete without a reference to such towns as Windsor, Ont., Hanover, Pa., Colfax, Wash., Palouse, Wash., Connersville, Ind.,

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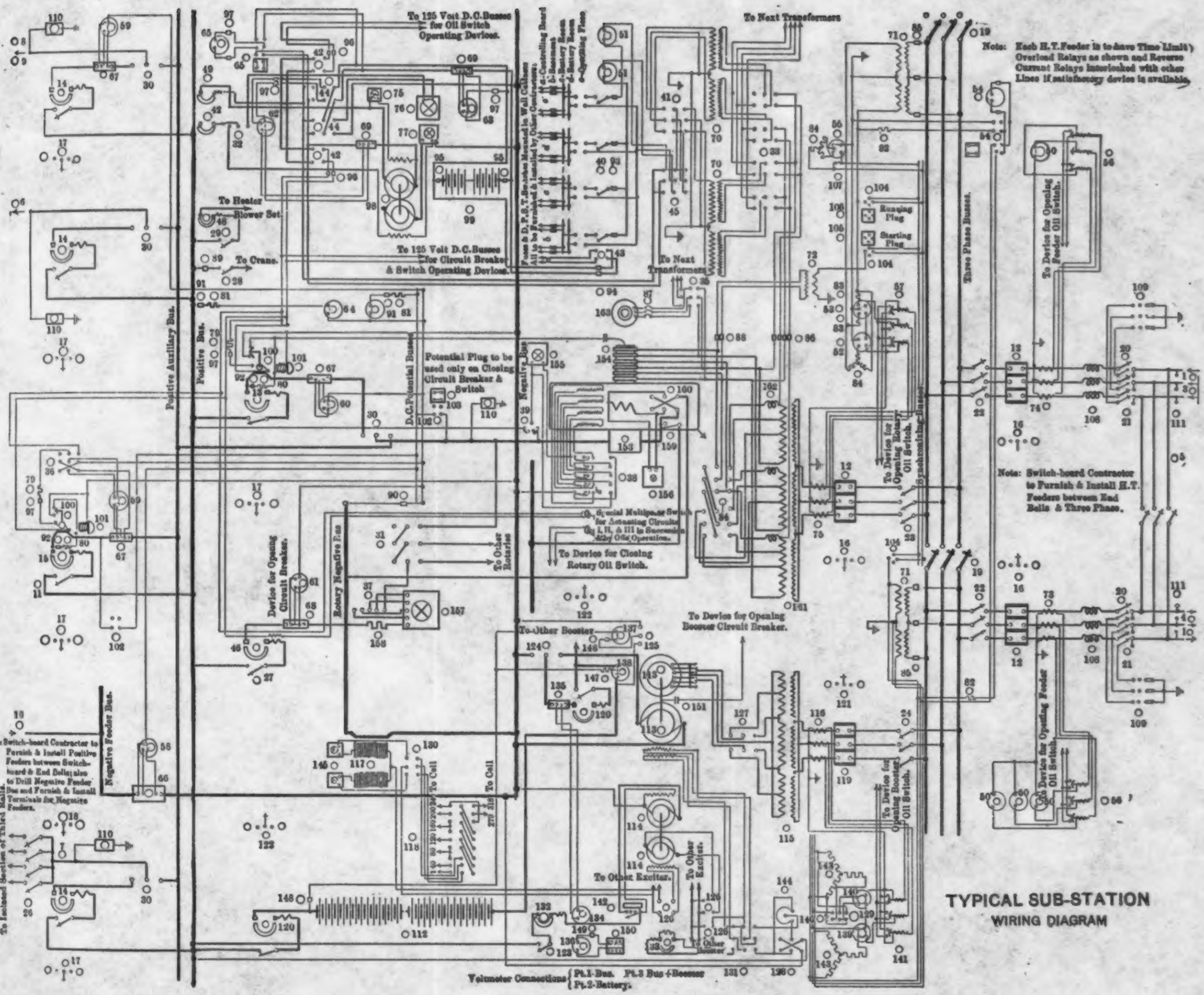

TYPICAL SUB-STATION
WIRING DIAGRAM

Rushville, Ind., Greensburg, Ind., Shelbyville, Ind., Napa, Cal., Vallejo, Cal., Exeter, Cal., and Rochester, N. Y., where single-phase systems are installed to-day. In all but Rochester, N. Y., the voltage in the trolley wires varies from 3 300 to 6 600 volts, not in restricted territory devoted exclusively to the terminals of the respective roads, or private right of way, but in the streets and highways. If high voltage brings with it high efficiency, smaller currents to be collected, smaller fixed charges, and in consequence of its physical attributes, lower operating costs, there can be but one argument against it. It is granted that the argument of safety is one deserving the greatest consideration. But is it not true that the greatest source in the agitation of this question of safety is prejudice? Every electrical engineer will agree that 100 000 volts can be placed on a trolley with perfect safety if beyond peradventure of a doubt two conditions are satisfied: (1).—That it is beyond the reach of the tallest man; and (2).—That the trolley wire will remain in its place. Mr. Murray.

It can be asserted that the first condition on all roads, as far as the public is concerned, is satisfied. As concerns the second requisite, on the 1 000 miles of high-voltage trolley that have been installed to date, the speaker has yet to learn of a citizen, not employed by the railroad, whose life has been forfeited on account of this form of electrification. These statistics will dictate the use of the high-voltage system, as they controvert the only argument that stands in its way to-day. This is the day of civic reform. Political control of appointments of public officers who are to regulate public service corporations will soon be a thing of the past. The States will see the advantage of the appointment of high-salaried engineers, whose integrity will be a guaranty that the practices of the past will be abandoned, and in consequence the railroads of this country will be urged to present arguments based upon facts rather than prejudices; and, while the City of New York and other cities may not yet adopt high-voltage trolleys, such men, when convinced that railroad companies can secure greater economies in their several operating departments by the use of the high-voltage trolley, will allow its use within the restricted zones included in terminal and right-of-way property.

Returning to the matter of the saving of $300 000 per annum, on account of the adoption of an electric engine designed for exclusive use on direct current instead of one which is operative on either direct or alternating current, and before referring to data of actual record concerning the economies of these two classes of engines, the speaker wishes to ask Mr. Wilgus why he compares the direct-current locomotive with the locomotive which is operative with either class of current? Why not compare the direct-current locomotive with the alternating-current locomotive? The author makes a comparison between two locomotives, one of which, by necessity, performs two func-

Mr. Murray. tions to its competitor's one. In spite of this, however, the inherent law which requires that a machine, in order to be a good alternating-current motor must necessarily be a good direct-current motor, has been demonstrated in the records of the electric meters installed on the New Haven locomotives for measuring the power while handling the trains in the New York Central zone.

While tests could be conducted to show that either locomotive would carry a given trailing load from the Grand Central Station to Woodlawn at the expenditure of a less number of kilowatt-hours than the other, such tests, unless conducted over a long interval, are valueless, but Table 5 is the record for the month of February, showing the energy consumption upon which the New Haven road is billed by the New York Central Company, and is of value.

TABLE 5.—Total Kilowatt Consumption for Electric-Train Service in New York Central Zone.

Date. February.	Direct current, hours.	Direct current, miles.	Total tonnage.	Passengers carried.	Number of commercial trains.
1	7 110	564	11 384	7 961	40
2	4 650	420	7 148	5 196	21
3	6 880	552	11 116	9 190	39
4	7 010	540	11 388	7 708	38
5	8 110	600	11 666	8 401	44
6	7 490	564	12 205	8 639	42
7	7 750	600	11 687	9 611	44
8	8 860	660	14 114	11 270	46
9	4 150	396	7 295	5 131	21
10	5 690	456	9 231	7 515	35
11	5 360	420	8 314	6 578	32
12	6 420	588	12 018	7 415	43
13	7 900	648	13 261	10 536	46
14	8 290	624	13 757	10 973	46
15	9 030	736	15 932	12 055	50
16	4 730	432	7 778	5 628	21
17	8 760	660	9 008	11 790	46
18	8 530	624	13 463	11 112	45
19	8 730	696	14 285	10 432	48
20	7 890	624	13 851	10 957	46
21	8 650	672	14 607	12 098	48
22	8 910	784	15 494	8 094	49
23	4 230	360	6 607	4 575	18
24	8 610	672	14 561	11 833	47
25	7 840	660	14 412	11 448	48
26	7 610	636	14 412	10 860	48
27	9 310	660	14 547	11 732	48
28	8 780	684	14 629	11 844	48
29	8 580	756	16 332	12 887	50
	215 860	17 388	354 399	273 469	1 197

Average watt-hours per ton-mile = 41.9.

The speaker does not wish to enter here upon a theoretical discussion of the rate of energy required to discharge a given schedule, but wishes to point out the fact, well known by all electrical traction en-

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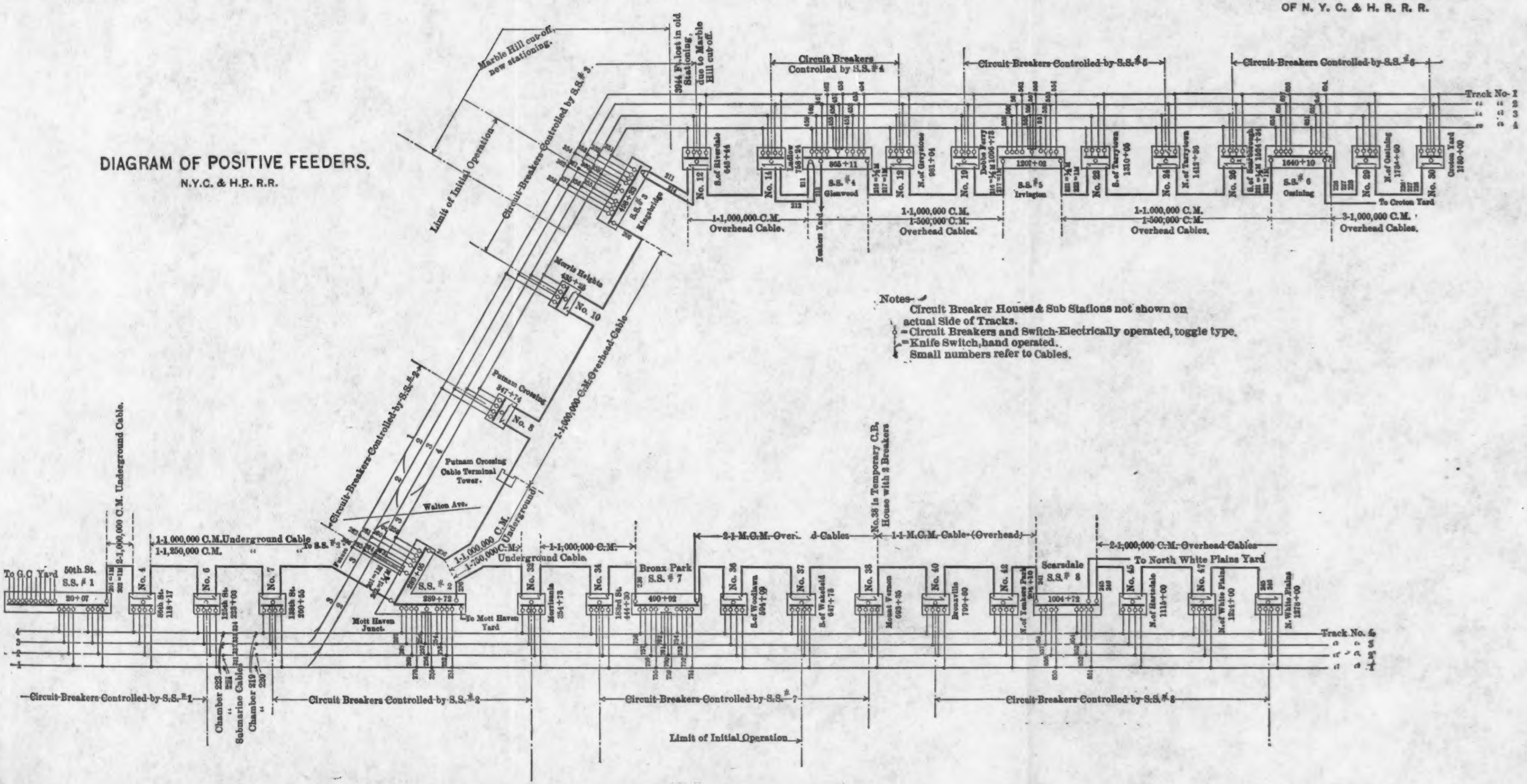

gineers, that slow-downs or stops not included in the regular schedule between any two points always increase the amount of energy required. Neither the committee of which the speaker was a member, nor the several others that followed in rapid succession, could arrive at a mutually agreeable conclusion as to an equitable rate the New Haven road should pay the New York Central for power, and it was seen, if power was to be purchased at the electric locomotive shoes, that the rates would have to be settled by an impartial committee. A firm of high standing in engineering circles was engaged by the two companies, and the question of delays and stops included between the Grand Central Station and Woodlawn, played so important a part in the consideration of a mutual basis of agreement between the two companies, that the New Haven road was requested to accept six slow-ups and five stops between South Mt. Vernon and the Grand Central Station, and six slow-ups between Grand Central Station and South Mt. Vernon; and the estimated rate of consumption, which that road was requested to accept as a determining factor for the amount of capacity that should be reserved for it in the Port Morris Station, was at the rate of 68 watt-hours per ton-mile. As shown in Table 5, the average was 41.9 watt-hours per ton-mile. The estimated amount is thus 62% greater than the actual amount measured, and it has occurred to the speaker that Mr. Wilgus may have also been mistaken about the relative amount of current taken by the two types of locomotives. Mr. Wilgus has given 33.9 watt-hours per ton-mile in his table of comparative tests of steam and electrical locomotives in switching and hauling service. These figures are in a manner a confirmation of his statement, and yet these represent figures in a test covering only two trains for two weeks. The New Haven figures cover an equipment necessary to the haulage of as many as fifty trains per day during a period of one month, and, in addition, are the basis of the New York Central charge for current. It should be added, also, that in actual test the speaker has seen the New Haven locomotives perform the same schedule as the one mentioned by Mr. Wilgus, at an energy rate of consumption not greater than the figures mentioned by him. It can be done. Mr. Murray.

In Mr. Wilgus's statement of a $300 000 saving per annum (which the speaker has reason to believe is zero instead of this large amount), there is no mention of the attendant fixed charge of the system producing the saving. In order to prorate properly the operating and fixed charges of the electrical distributing system jointly used by the two companies between Woodlawn and Grand Central Station, a distance made up of 12 miles of four-track railroad, it was necessary for the New Haven Company to audit the construction charges of the New York Central Company. This duty devolved upon the speaker. Exclusive of power-house and motive power (and, incidentally, the New

Mr. Murray. Haven alternating-current-direct-current locomotive cost less than the New York Central locomotive, notwithstanding the continuous capacities of the two machines are very nearly the same), the cost per mile of electrification is in the ratio of 5 to 1. It is true that much of this cost is made up of land, which was necessary for sub-stations, and thousands of pounds of copper, but it should be remembered, in comparing it with the single-phase system, that in the latter these requirements are practically dispensed with. Sub-stations are reduced to zero, and copper to a minimum, and so the speaker thinks that in mentioning this increase of power required to operate New Haven locomotives, this attendant factor of five times the fixed charge should be, at least, mentioned.

As much in importance, after considering the fixed charge of an installation, is the cost of operation. Again, in virtue of the necessity of the close co-operation of the two companies, in so far as the auditing of accounts is concerned, figures appertaining to this interesting subject necessarily passed through the hands of the speaker. It was found that for 12 miles of four-track road there is a charge, for maintenance and operation, of five times the amount the New Haven road pays to maintain and operate 21 miles. Although he has the exact figures covering the disbursements necessary to the installation of each form of distributing system, he has not felt justified in presenting these figures at this time. However, the ratios mentioned serve the same purpose.

In conclusion it should be stated that the principal object of the New Haven engineers has been simplicity. True, the use of the direct current on the rails over which their locomotive had to operate was a sort of kink in the wire, but they were not responsible for this, nor has it carried any especial terror to their hearts, and had it not been superimposed upon them, like the other parts of the system, the control in their locomotives would have been simplicity itself.

Mr. Wilgus has spoken of the violent fluctuations of the load on the power-station and sub-stations, which are corrected by the use of storage batteries. On the twenty-one miles of the New Haven road there are no storage batteries, but, due to the high efficiency of transmission and the prompt regulation of the generators at the power-station for fluctuating loads, even at the western terminus, the most distant from the power-station, the voltage seems to be practically as stiff as at Cos Cob.

In yards, a light but strong cross-catenary form of construction is readily applicable to this branch of electrification. The question of dodging the third-rail no longer confronts the yard hand. There is 8 ft. of good air between the trolley wire and the tops of the freight cars, thus providing clearance for the tallest man.

The speaker's argument has been based upon official data, for the

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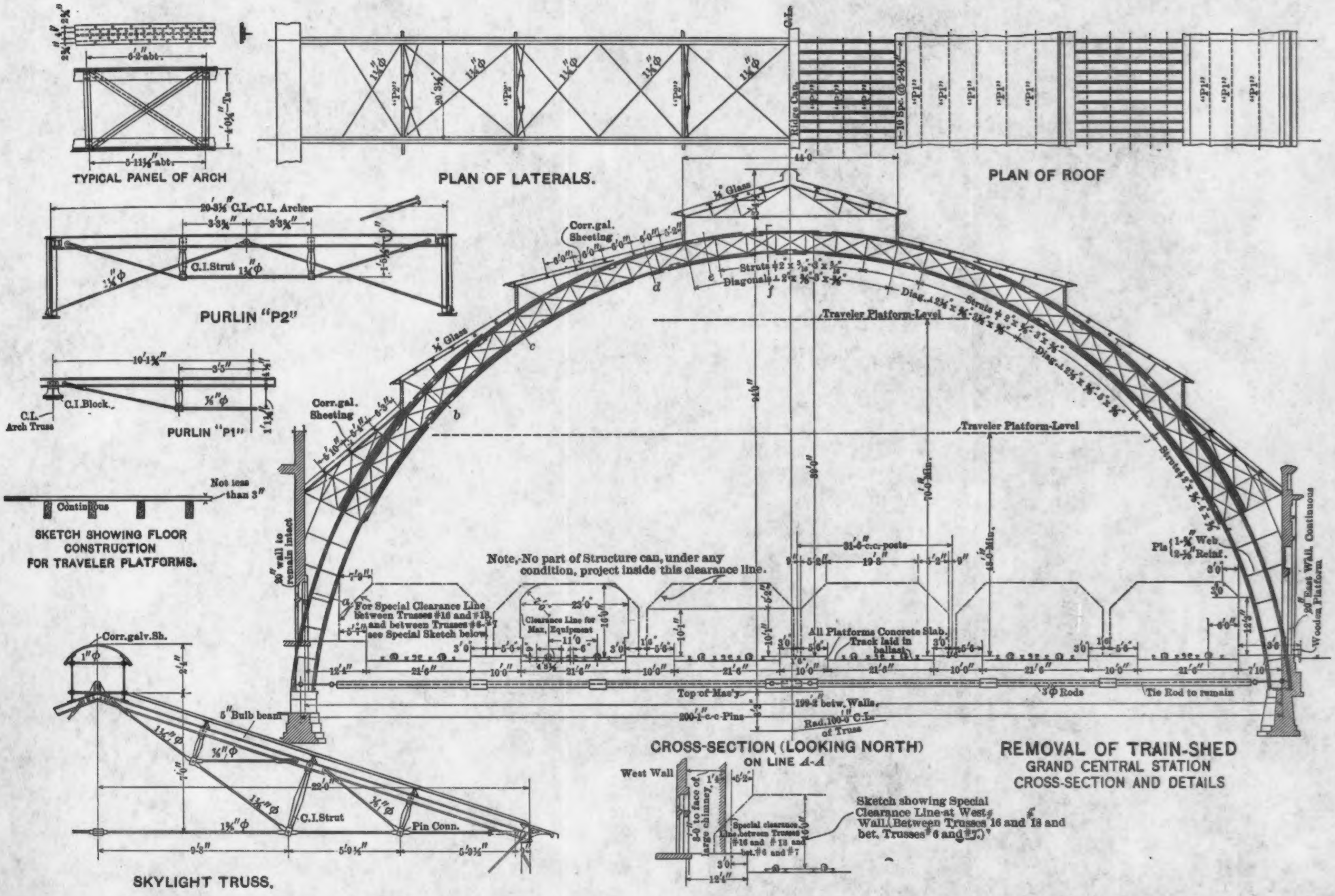

REMOVAL OF TRAIN-SHED
GRAND CENTRAL STATION
CROSS-SECTION AND DETAILS

purpose of controverting the statements made against the alternating-current system, and he is willing to go on record that the near future will see the high-voltage distribution system, with its attendant alternating-current locomotive, propelling trains from and between the terminals of cities where the density of traffic is sufficient to warrant the increased fixed charges for electric traction. Mr. Murray.

GEORGE A. HARWOOD, M. AM. SOC. C. E. (by letter).—This paper, for the period which it covers, is so complete that little can be added. Some features of the improvement, however, which have been taken up since the paper was prepared, may be of interest. The most important of these is the removal of the train-shed of the Grand Central Station. Mr. Harwood.

It was originally contemplated that traffic would be transferred to the new east side station before it became necessary to remove the old train-shed. This was prevented by the increase in traffic and the progress of the excavation. Last fall, therefore, it was decided to remove the shed while the through passenger trains of the New York Central and the through and local trains of the New Haven continued to use the old station.

It is proposed to remove all of the shed north of the passenger waiting-room. The length of this portion is about 600 ft., and consists of wrought-iron arches, the material for which was imported from England and erected in 1870. The arches are built in the form of a truss, the section being about 4 ft. from back to back of chord tees. They have a span of 200 ft. 1 in., from center to center of pins, with a clear distance of 85 ft. from the top of the platform to the under side of the arch. The bottoms of the arches are tied together under the tracks with 3-in. rods. The distance from center to center of arches, longitudinally, is 20 ft. $3\frac{1}{2}$ in. The details of construction are shown on Plate XXIX. There are about 1 350 tons of wrought iron, 350 tons of cast iron, 90 000 sq. ft. of corrugated-iron roofing, 60 000 sq. ft. of glass, and 530 000 brick to be taken down, loaded into cars and removed from the terminal without interference with the regular business.

To accomplish this, and reduce to a minimum the possibility of accident by falling material, it was decided to erect a traveler, the outlines of which would conform to the general contour of the train-shed, spanning all platforms, with heavy floors extending the entire width of the shed. The supports for the traveler, details of which are indicated on Plate XXX, rest on the five intermediate platforms, and are carried on heavy cast wheels which roll on standard 100-lb. rails. The load on each platform, including the weight of two of the train-shed trusses, is 200 tons, this being distributed over the entire width of the platform by ties supporting the rail, and covered with temporary planking so as not to interfere with the use of the platform for regular business. The traveler contains 370 000 ft., b. m., of lumber, 65 tons of bolts and washers, and 33 tons of plates and castings. It has a length

Mr. Harwood. of 65 ft., which will permit of blocking up two of the shed trusses on it at the same time. It is equipped with six stiff-leg derricks which are operated by two engines. Flaps are hung on the sides, from the first platform level, to protect passengers on the outside station platforms. The south face is boarded over, forming a false end for the train-shed as the traveler moves south. The material was all framed at the company's yard at Harmon. Portions ready for erection were then brought into the station and set, up to the elevation of the first floor, at night. After this had been placed, the remainder of the traveler was erected during the day, the members being brought in and lifted to the first-floor level at night.

Fig. 11 indicates the general method of procedure. As fast as the traveler is moved south it is followed up with temporary wooden canopies, thus subjecting the platforms to a minimum amount of exposure. As soon as the traveler has been moved to a new position and blocked up, two train-shed trusses are blocked on top of it, and the corrugated-iron roofing, glass, skylights, and purlins are removed. The northerly truss is then cut into eight sections by using hack-saws and knocking off the rivet heads at the joints. The derricks then place these sections on the traveler platforms, all this work being done during the day. The night gang loads the material from the platforms in cars placed on the passenger tracks under the traveler.

The traveler is moved by jacks, two 15-ton jacks being placed on each platform, and each being operated by two men at signal so as to maintain a uniform movement. Fig. 1, Plate XXXI, shows the manner of applying the jacks. The work is done in units of 40 ft., and it requires 5 hours to move the traveler this distance. Now that the north portal and the first few northerly bays are removed, it is expected that the average progress will be about one truss for each 4 days.

The taking down of the north portal was the most delicate part of the operation, on account of the necessity for cutting away all connections between the portal and the train-shed before the work of demolishing could begin. The north end of the traveler was constructed with beams projecting 5 ft. beyond the face. These were pushed through the window openings, or through openings cut in the metal sheathing, and the entire portal was lashed to the traveler. Wooden troughs were constructed at the various platform levels so as to prevent loose metal from falling on the tracks below. Fig. 2, Plate XXXI, is a progress photograph of the removal of the north portal.

Fig. 3, Plate XXXI, a progress photograph giving a general idea of the work, was taken after the first two trusses south of the north portal had been removed, and shows the first canopy posts supporting the ends of the old canopy which existed north of the train-shed.

Fig. 4, Plate XXXI, shows the southerly face of the traveler, and,

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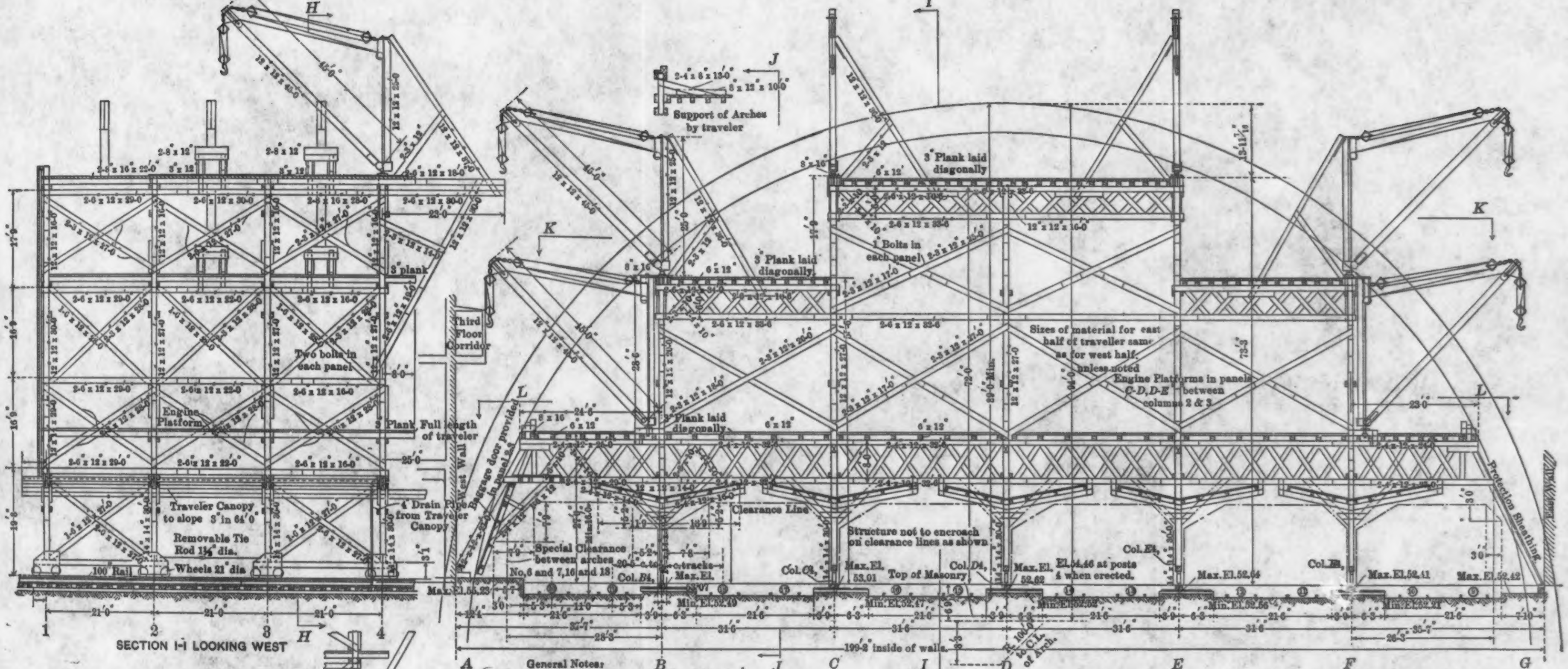
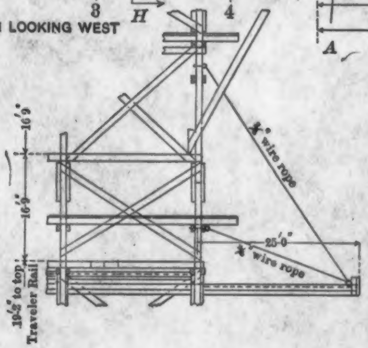
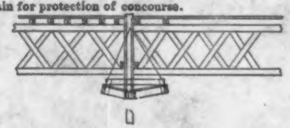




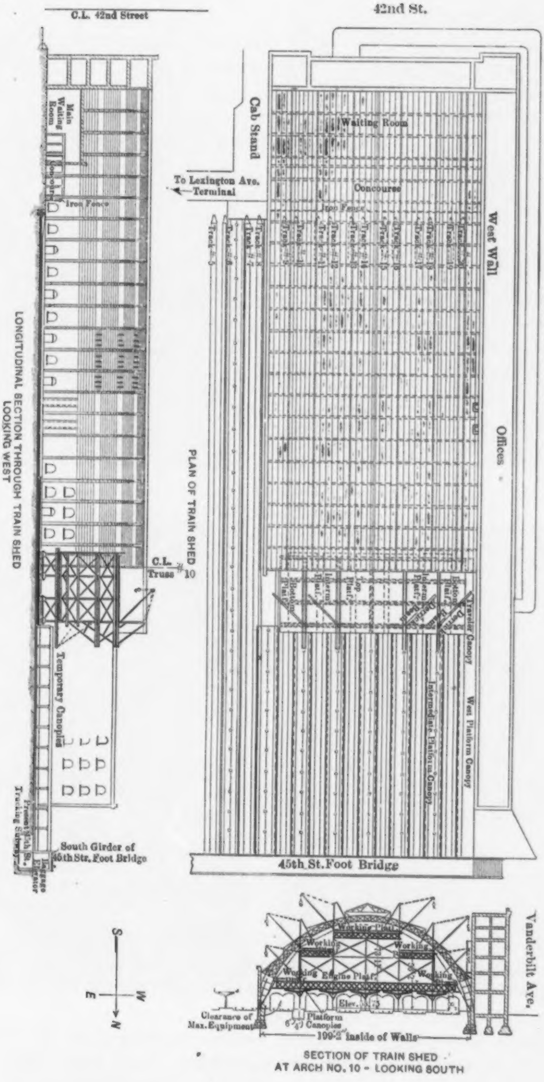


General Notes:
Track to grade down to south 1" in 100'
Length of material given here is ordered length only.
Flooring short-leaf yellow pine. Sheathing and louvres, Spruce
For all other work use inspected long leaf yellow pine.
1" dia. bolts used throughout unless noted otherwise.
Platforms to be pitched to drain to south face of traveler.
Canopies to be pitched to drain north.
Traveler to have two coats of paint after train-shed has been dismantled in case it is decided that traveler will remain for protection of concourse.

Max. dead load on Col. 60000 lb. Cols. B2, B3, F2, F3
Max. live " " " 88000 lb. Cols. D2, D3
Max. Vertical Wind load on Col. 41000 lb. Cols. C1, D1, E1
Max. Total load on Col. 150000 lb. Cols. C4, C1, D1, D4, D2, D3, E4, E1
Live Load on platforms 100 lb. per sq. ft.
Wind Load 20 lb. per sq. ft.

CROSS-SECTION OF
TRAVELER FOR REMOVAL OF
TRAIN-SHED
GRAND CENTRAL STATION

Mr. Harwood.



GENERAL METHOD OF PROCEDURE IN REMOVING TRAIN-SHED

Fig. 11.

Mr. Harwood. through the openings, the canopy construction following the movement of the traveler may be seen.

Fig. 12 shows the details of the temporary canopies. These are of the butterfly type, with a large overhang so as to provide maximum protection.

The preparation of the detailed plans and the execution of the work are under the charge of Mr. J. L. Holst, Engineer of Structures.

DETAILS OF TEMPORARY CANOPIES,
GRAND CENTRAL STATION.

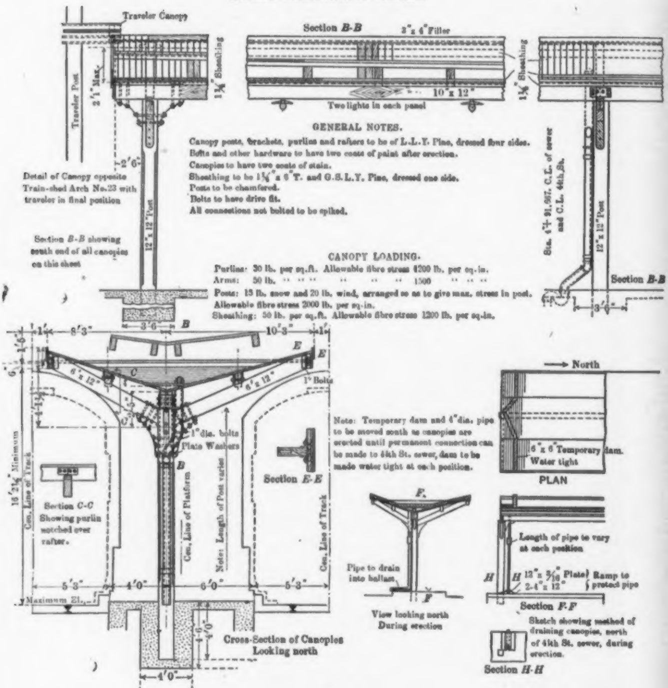


FIG. 12.

Mr. Potter. W. B. POTTER, M. AM. SOC. C. E.—This paper presents a comparison of steam and electric operation in a very interesting and comprehensive form, and the speaker wishes to express his appreciation of the valuable information given therein.

Reliability of service under electric operation is very properly regarded as a matter of first consideration. The installation of duplicate power-houses and storage batteries, considered as an insurance,

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FIG. 1.—MANNER OF APPLYING JACKS, IN MOVING TRAVELER.

FIG. 2.—REMOVAL OF NORTHERN PORTAL OF TRAIN-SHED.

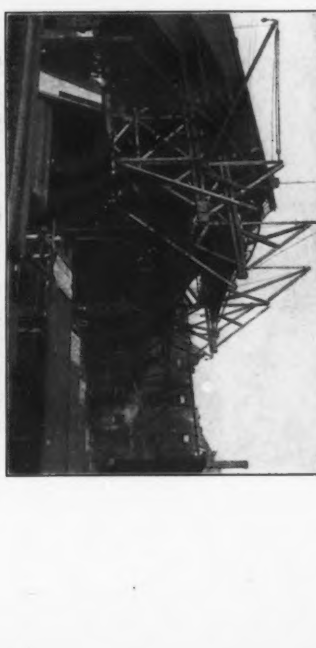

FIG. 3.—REMOVAL OF TRAIN-SHED. PROGRESS VIEW.

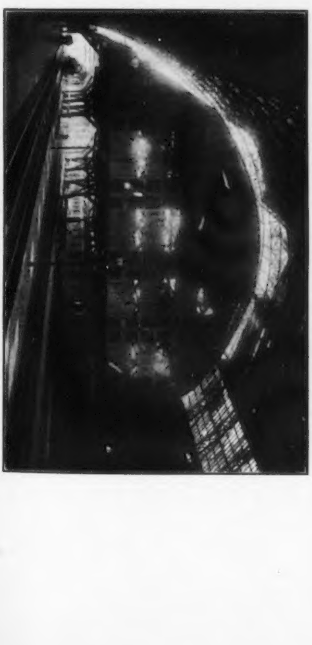

FIG. 4.—SOUTHERLY FACE OF TRAVELER.

Mr. Potter.

may have been well justified by the importance of the traffic. The speaker believes, however, that the individual generating units will be found to be a sufficient reserve to ensure reliable service, and that the second power-house will ultimately be regarded as simply a plant for additional load.

The type of electrical equipment for the rolling stock was also selected with reference to reliability. Neither duplicate power-stations nor batteries would be of avail to clear a block caused by trouble with the rolling stock. The New York Central locomotive was designed to handle a 550-ton train, including the weight of the locomotive, and the heavier trains were to be double-headed. Double-heading, however, has not been found necessary, a single locomotive in ordinary service having frequently drawn an 800-ton train, that is, twelve or fourteen Pullman cars.

Mr. Murray has remarked that if one were constantly starting one would never get anywhere. This is equally true if one does not start at all. A successful locomotive must be capable of starting the train which it has the horse-power capacity to haul at the required maximum speed. The New York Central locomotives have about 137 000 lb. on the drivers, and, with sanded rails, can exert a draw-bar pull of 45 000 lb., and, while this is in excess of the usual requirement, it does ensure starting under abnormal conditions. The diagram, Fig. 2, given by Mr. Wilgus, shows the normal acceleration and speed of the locomotives with different weights of trains.

For the through trains on the New York Central, the locomotive was unquestionably the proper motive power, and for the local service either a locomotive or multiple-unit cars might have been used, but the selection of the latter was undoubtedly wise. As each multiple-unit car can be moved independently, yard movements are facilitated, and also the making up of trains which are leaving on close headway, and, as either end of the train may be the head, economies are effected in the way of extra locomotives, track space, and switching that would be required in the case of locomotive operation.

Multiple-unit equipment, further, provides for an amount of power proportional to the requirement. The speaker doubts whether it is appreciated that an Interborough express train has the same motor-power equipment as a New York Central locomotive, and, with over 50% more weight on the drivers, the Interborough trains accelerate at nearly the slipping point. The equipment of each car with motors suitable for the service makes it possible to maintain the schedule with heavy trains with the same certainty as in the case of a single car.

The speaker has been particularly interested in the author's comparison of the cost of steam and electricity, as applied to locomotive service. His figures for work of this class, as the estimates fre-

Mr. Potter. quently indicate, show that there is a saving in favor of electricity which will be found in reduced incidental expenses rather than in the cost of direct operation.

There are some points in Mr. Murray's discussion with which the speaker does not agree. The reliability, initial cost, and cost of operation, for the service between New York and Stamford, are all in favor of direct current. This will be more especially true if multiple-unit trains are operated in the local service. The complexity incident to alternating-current and direct-current operation, and the relative cost, as influenced by the number of trains under the particular conditions in this instance, introduce features unfavorable to the alternating-current system. The speaker, however, does not wish to be misunderstood, or to be regarded as not favoring the alternating-current system under conditions more favorable to its application. He thoroughly endorses what Mr. Henderson has said in regard to the importance of studying the local conditions as affecting the general scheme of electrification, but believes that greater density of traffic usually insures a greater uniformity of load, so that the common expression, "density of traffic is favorable to electrification," is usually true.

Mr. Wilgus and his associates are to be congratulated on their success, and for the satisfactory service which they have rendered the public in the work they have done.

Mr. Sprague. FRANK J. SPRAGUE, M. AM. SOC. C. E.—The Society is to be congratulated, and Mr. Wilgus complimented, upon the presentation of one of the clearest expositions of the results obtained in an electrical equipment of great magnitude, under particularly trying steam-railroad conditions. It is a concise and definite statement of facts, where real comparisons are possible, made in plain terms, and within the reach of such railroad men as are not trained electrical technicians.

With the view taken by Mr. Henderson, the speaker wishes to express cordial agreement, as it is one which he has spent a number of years in voicing, namely, that the wisdom of the application of electricity to the equipment and operation of a steam railroad—and, it may be added, the method of such application if adopted—is a problem individual to the needs and conditions existing on that particular road, and cannot be determined by any general statements of real or fancied gains in fuel economy. Nor is it sufficient that economies, of whatever nature, promise a fair interest on the new investment required, for, if this were all, there would be no excuse for the adoption of electricity, because the existing investment would have derived no advantage, and every railroad man knows that there are ways of investing money in railroad improvements which will result in much more than a return sufficient to pay interest on the new capital—a fact not without vital influence at a time when money seeks the seclusion of the tin box and the stocking.

At the present moment the field of electric application is more apparent where trunk-line terminals and suburban service are concerned, for here there are certain favorable conditions conducing to success, such as density of traffic, fair load factor, and reasonably uniform traffic conditions and train movements, while public sentiment favors the abolition of smoke and steam, and suburban populations demand more rapid and frequent train service. Mr. Sprague.

In addition, there are mountain roads where onerous conditions are imposed by local difficulties or connecting divisions, and these, in spite of an irregular and widely varying traffic, may advantageously consider electrical operation. In both classes, however, the key-note is capacity, with an eventual gain in unit and system economy.

Up to the present, the most conspicuous example of terminal and suburban equipment is that so well described by the author, and with regard to this Mr. Murray states his belief that a mistake has been made, and that if the decision as to the most suitable equipment were to be made now "a majority of the best informed electrical engineers of America would to-day cast their ballot in favor of single-phase electrification."

Just here, and without expressing any opinion as to the merits of that system—and there are many valuable arguments in its favor for some classes of roads—the speaker must record his judgment that this statement is entirely wide of probability and fact, and, basing his conclusion upon the comparative results shown by the competitive systems now in operation on the New York Central and New Haven roads, must record his conviction that if confronted to-day with like actual conditions, not only would the majority—and he trusts all—of the members of the New York Central Electrical Commission, but any other competent board representing sound electrical engineering and conservative railroad practice, make the same general decision as originally made, modified possibly in degree of potential adopted and some details of equipment. Furthermore, it is quite possible that the somewhat extraordinary degree of insurance of operation would not now be decided upon, but it must be remembered that, to a certain extent, much of what was adopted was based upon the idea of providing insurance at first and utilizing capacity later.

In the development of the equipment for the New York Central, there were some matters of greater or less novelty concerning which some risk had to be taken, and among these there may be mentioned the vertical turbines, the particular type of locomotive, and the under-contact protected third-rail. There were ample criticisms and many predictions of failures as to all three features, and from many quarters; but it is with some degree of complacency that the speaker recalls his own connection with this development, and the practical results attending the adoption of these particular features.

Mr. Sprague. There have been no "frightful accidents" with the turbines, and the locomotive has shown a capacity and reliability simply amazing, for it has been called upon for a hauling capacity of nearly 100% in excess of its guaranty. On some recent trials it has run more than 650 miles on consecutive days, stopping and starting within 6-mile limits, and has made nearly 700 miles within 24 hours under these conditions. The machines are not perfect, and, undoubtedly, some minor changes will ultimately be adopted, but it is instructive to note that, in the year and a half of operation, the makers have spent probably not more than $4 000 in changes on the entire equipment, and not one dollar of this has had to do with the motors or their design. In that period the electric service has increased up to a present maximum of about 300 movements a day by electric locomotive and by multiple-unit train, and in all that time there have been recorded but two electric locomotive failures on the track, where the locomotive could not pull itself. Its single-phase competitor, in one-third of that time, on 13 miles of road, and with a maximum of 60 train movements, has a record of at least 40 failures. If it had not been for the New York Central's electric locomotives, or multiple-unit trains, the effect of these failures would have been more pronounced.

Undoubtedly, the causes of some of these failures will be removed, and a better record will be shown by the single-phase machines, but it will be some time before there can be any final comparison between these two systems. Two facts, however, even now stare one in the face: The first is that the speaker's predictions (made some time ago) as to the relative capacity, have been borne out in practice, for while single direct-current locomotives have thus far pulled any load which has been put behind them, it is customary to use two single-phase locomotives whenever the trailing load exceeds six suburban cars or six cars of moderate weight. The second is that, in spite of the adoption of locomotive operation only, operative demands will eventually make necessary the use of multiple-unit trains for all suburban service out of great terminals; and, when this latter is attempted under the conditions which exist, the net result of the operation of such trains on combined direct and single-phase divisions will be absolutely disappointing when compared with the operation of multiple-unit trains on the direct-current system alone.

A third feature, which was a matter of some concern, was the under-contact third-rail, in the development of which the speaker had the pleasure of co-operating with Mr. Wilgus. It was possibly somewhat more costly to install this third-rail, under existing conditions, and it has been more or less costly in its up-keep. The speaker has made an effort to secure some comparative costs of maintenance of this rail, both on the New York Central and in other localities. The

local conditions on the New York Central are unusual in character, Mr. Sprague. and the accounts are complicated more or less by the costs of new construction, so that they cannot at the present time be given with any degree of reliability. One special difficulty has developed, to which attention should be called, which, however, is the fault of a collateral feature of the general equipment. A third-rail, whether of the top- or bottom-contact type, and especially where there is much special work, will operate most satisfactorily when the contact shoe is maintained within reasonable limits of operation. Unfortunately, the contact shoes on the New York Central locomotives are not carried, as they should be, upon the frames moving with the pilot trucks, but upon the superstructure above the equalizing springs. The result is a horizontal and vertical movement probably fully three times as great as would exist with another possible method of mounting. This has been the cause of some trouble at side inclines and elsewhere.

The speaker has reports of very different character from some other roads, where this particular difficulty does not exist, and where the operation is strictly normal, and comparable with other systems of working conductors. On the West Shore Railroad, operating for the 8 months from July 1st, 1907, to February 29th, 1908, with a monthly average of 77 204 miles, the official reports give the following averages per month during this period:

Material	$163.26
Labor for repairs	18.32
Labor for inspection	297.71
Total	$470.29

As this equipment covers 105 miles of track, the total cost chargeable to third-rail is $4.56 per month per mile.

The Philadelphia Rapid Transit Company has 12 miles of this rail, and the chief engineer reports an actual car mileage of 1 500 000 miles for 1907, and states that:

"The total cost of up-keep during 1907 has been practically nothing, as but two insulators required changing, and these were probably cracked when installed. The covering will probably require painting during 1908."

Of course, in time there will be opportunity to make direct comparison between the cost of up-keep of this type of third-rail and various types of overhead construction, and, concerning the latter, the speaker is inclined to think there will be some developments, but it is interesting to note one comparison, that of the maintenance of a top-contact unprotected third-rail and an overhead trolley of the usual class on one of the most important electrical railroads in America,

Mr. Sprague. operating nearly 128 miles of third-rail and nearly 20 miles of trolley wire. In this particular instance, recent reports show a ratio of 3.8 in favor of the third-rail.

On the subject of the adoption of systems, and as illustrating how unsafe it is to predict what will be done in any particular instance, it is but proper to call attention to the fact that the electrical engineer of the Government railways in Belgium—they are all owned by the State—came to America a few months ago, from close proximity to alternating-current developments, and with some prejudice in favor of them. He had ample facilities for inspecting equipments in America, and on his return he reported to his government squarely in favor of using direct current and third-rail for the first equipment put in by the Belgian Government, and orders for a part of this equipment were recently supplied from America.

The speaker has also recently received from one of the foremost electrical engineers in England, Mr. Parshall, who has been identified with a large amount of electrical work there, is very familiar with conditions abroad, and follows developments there very closely, a letter which reads as follows:

"You have heard a great deal of alternating current development on this side of the water; I beg to assure you that they are all interesting from a laboratory standpoint, not one would meet the conditions of American practice; they are instructive as telling what not to do rather than what to do."

The speaker does not wish to prejudice the cause of the single-phase system. He seeks but the truth with regard to any equipment; there are cases in which he believes that that particular system may be used with advantage, but he holds unalterably to the view that the very best interests of electric railway development require every possible advance in either system, and that a hide-bound adherence to any one cannot but result in adverse developments.

Mr. Stott. HENRY G. STOTT, ESQ.*—Mr. Wilgus gives certain reasons for duplicating power-stations and transmission lines, but a stage in the art has been reached where there is no longer any very strong argument for duplicating power-stations. The speaker's experience, covering quite a number of years with power-plants, is that the greatest danger now is not from the apparatus, but from the men who operate it. Take, for example, the two largest power-stations in New York City, each of which is giving out about 75 000 h-p., morning and night. In the past six years of operation the speaker can recall only two shutdowns which were due to the power-plant, and these were of very short duration, from 10 to a maximum of 20 min. It is obvious that the duplicate power-plant would not help such a situation, because, unless

* President, American Institute of Electrical Engineers.

all the boilers were fired up and the units were turning over continuously, at least 40 min. would be required to get them into service, by which time the power-station would be in operation. Mr. Stott.

In regard to the transmission lines, the power-stations necessarily must all be connected to a common system, otherwise, the copper installed would be wasted, and the rotaries in the sub-stations would have to be started over again, unless they were connected.

A few days ago there was a shut-down on the Manhattan system. The trouble was in the transmission lines, and was caused by one of the steam heating mains in the city destroying the insulation. If there had been a dozen power-houses, instead of two, there would have been the same trouble; and it can safely be said that it is a great deal better now to "put all our eggs in one basket and watch that basket, than to put them into separate baskets," in view of the fact that the human element is now the most dangerous one.

The operating and fixed charges are certainly less for one plant than for two. In working out power costs it is just as necessary to take into consideration the fixed charges as the operating costs. The fixed charges in many cases are greater than the actual operating and maintenance cost. It is a very poor plant to-day which cannot operate at less than 0.7 cent per kw-hr., that is, for operating and maintenance charges only. Fixed charges on a load factor of 50%, which is about the best load factor that railroads can expect, would be at least equal to that. Now, if the load factor is less, naturally the operating charges become of less importance, and the fixed charges of greater importance. In the plants described in the paper there is apparently going to be 100% spare apparatus. The safe over-load capacity of modern generators and turbines is usually given at 50% above rating, so that, even if the whole apparatus is being used, there is always a reserve capacity of 50%, making in this case a reserve of 200%; but, in considering peak loads, such as all railroads get during the morning and evening, due to the movement of suburban trains, a very much smaller load factor must be considered. In this case it is most important to keep down the fixed charges, which, under these conditions, may become three or four times as great as the operating charges.

William J. Wilgus, M. Am. Soc. C. E. (by letter).—As anticipated, this discussion has brought out many points which will go far toward solving the modern problem of electrifying steam railways. Mr. Wilgus.

Reference is made by Mr. Henderson, Mr. Gibbs, and Mr. Francis to the large collateral expenditures that have a bearing on the comparative cost of operation by steam and electricity; but one should not lose sight of the fact that such contingent expenses, in the installation under discussion, are a necessity entirely apart from the question of electrification. For instance, the enlargement of the Grand Central Terminal, four-tracking, elimination of grade crossings, and similar

Mr. Wilgus. items, are required for handling a growing traffic properly. Indeed, it is true that most of these improvements are only possible with a change of motive power; but that is an added argument in favor of electricity. In other words, if there had not been the desire and necessity of radically enlarging the capacity of the railroad, it would have been possible to have made the change from steam to electricity, with a resultant material but insufficient increase of capacity, without incurring any expense for other improvements.

On the other hand, there are many places where the installation of electricity, with its superior operating advantages, will not only save in annual cost of operation, but, in addition, obviate the undertaking of expensive improvements that with steam would be necessary for increasing the capacity of the railroad.

Therefore, while it is proper to concede that the use of electricity on the New York Central has invited the undertaking of other large expenditures for greatly increasing the earning power of the railroad, it is not just, in comparing the cost of steam and electric operation, to charge against the latter fixed charges other than those made necessary by the change of motive power alone.

The proof of the wisdom of making these collateral improvements must rest on later developments of traffic, when the entire scheme has been completed.

Mr. Henderson's analysis of the relation of average to "peak" traffic is very interesting, and is borne out by an application of his reasoning to the New Haven Company's installation, had direct current been used between Woodlawn and Stamford. As shown below, the annual propulsion current requirements at the Cos Cob power-station bus-bars is, say, 18 500 000 kw-hr. The power-station installation for propulsion current is assumed to be 13 500 kw-hr. for direct-current operation. This would make the ratio of power installation to average train requirements 6.4, the cost of electric operation being slightly lower than by steam. Probably a ratio of 6 would represent an equality of cost of operation by steam and electricity under New York Central and New Haven conditions, which would agree closely with Mr. Henderson's conclusions, if he had omitted the portion of the power-station installation not intended for propulsion purposes. Of course, on both roads, increase of passenger traffic and the later handling of freight traffic, labor-saving machines, and yard switching by electricity, will raise the average load and produce still better results.

As intimated by Mr. Gibbs, it is perhaps too early to forecast with positiveness the saving of electric operation over steam in the various installations made to date, but the many misstatements to the contrary that have appeared from time to time in technical discussions certainly point to the wisdom of throwing some light on the subject before the means disappear of making true comparisons under the condi-

tions. Regarding locomotive repairs, there is no doubt that changing of types and details will ensue, and the cost will be chargeable to repairs; but such changes are also constantly occurring in steam practice. The obsolete machine is simply relegated to a less exacting service. It is true that duplicate facilities must be maintained for handling the motive power at interchange terminals; but there will soon be no occasion for expensive facilities for handling suburban steam locomotives, as their electric substitutes will require small inspection sheds only, and the relatively small steam locomotive plants for through trains will be removed from costly New York City real estate to outlying cheap lands. As Mr. Gibbs states, there is difficulty, on electrified steam roads, in obtaining the desired full mileage from electric rolling stock. However, where the traffic is dense, and reasonable attention is paid to this feature by the operating department, it is believed that reliance may be placed on the results outlined in the paper. Mr. Wilgus.

Mr. Waitt's reference to European practice, based on his personal investigations abroad, is both interesting and instructive. There is the frequent tendency among American engineers to urge the adoption of foreign practice unsuited to American conditions.

The remarks of Mr. Lewis and Mr. Francis point to the desirability of a future paper dealing with the New York Central improvements from the "static" standpoint, but this, of course, cannot be done until the work has more nearly approached completion. Mr. Katte and Mr. Harwood briefly touch on some of the details.

Mr. Brinckerhoff's experience with elevated railway service gives much weight to his comparison of the operating costs and efficiency of steam and electric motive power, especially the item of maintenance, about which some question has been raised.

As remarked by Mr. Sprague, if the question of electrification is to be approached from the standpoint of a fair return on the new investment required, something beyond the ordinary rate of interest must be offered to the investor. While the electrification of the New York Central was undertaken for reasons apart from possible economies of operation, as will be shown below, the prospective savings are such as to promise a return of not only the ordinary rate of interest on the capital invested, but also an additional amount for the stockholders, the aggregate of these two items being estimated at about 9% on the additional capital required for electrification. The cost of maintaining the third-rail in the initial zone is comparatively high, not for the reasons given by Mr. Sprague, but because of the many minor adjustments, alterations of tracks, and close inspection, all of which are incident to the newness of the installation.

Both Mr. Stott and Mr. Potter express the belief that reliable service may be amply insured without the precaution of a second generating station. Referring to the reasons given for duplicate power-sta-

Mr. Wilgus. tions (page 77), it will be noted that two power-stations were decided upon:

> "each with sufficient capacity, utilizing its spare unit, and working 'overload,' to carry the entire demand of the service at the rush hours, should the other fail."

Had one power-station been decided upon, an additional unit would have been required for spare purposes, making 25 000 kw-hr. instead of 20 000 kw-hr., from which it will be noted that instead of 200% spare apparatus in the two power-stations, there is but 60%, including the 50% over-load capacity of the generators. Therefore the question of the wisdom of installing duplicate power-stations hinges upon the necessity of having this 60% excess capacity at the initial stages of the service, and the additional expense attendant upon the operation of two power-stations instead of one. The Electric Traction Commission considered these features thoroughly, and concluded that the company was justified in this additional expense for the reason that the geographical location of the two divisions, with their possible future extensions to the north, was such as to make unwise the adoption of but one station for both, located at a remote point and subject to the chance of such injury to itself or the connecting transmission lines as to make possible a long-continued interruption of train service. For instance, should a single power-station or its connecting transmission line suffer serious injury from rioters, strikers, accidents on adjoining property, or any other contingency sufficiently serious to place the power-station out of commission for a long-continued period, as was experienced on one of the English railroads which was electrified some years ago, a return to the use of steam locomotives would be imperative. The contemplated future electrification of freight service within the electric zone, including the terminals on the west side of Manhattan Island, the electrification of all or a portion of the Putnam Division, and the utilization of company current for lighting, yard switching, labor-saving devices, etc., promise a reasonably early use for the excess capacity of the power-stations sufficient to justify its expense for these insurance purposes during the early stages of electric operation. The location of the two stations, in better relation to the load centers than would be possible with one station, offers a saving of transmission losses that tends to compensate for the extra cost of operating two power-stations over one. That the company anticipates early need for the excess capacity is shown by its recent decision not to accept a proposition for its purchase or lease by outside commercial interests.

It will be interesting to note here that the cost of the power-stations was very low—less than $90 per kilowatt of capacity.

In the paper there was no intention of raising any issue with the representatives of the New Haven Company because that company saw fit to select a form of electrification different from that adopted

by the New York Central. The paragraph first quoted by Mr. Murray had reference to the proven wisdom of adherence to the chosen type of direct-current equipment, despite the urging by a manufacturing company upon the New York Central of the alternating-current-direct-current apparatus; and in no manner reflects on the use of the alternating-current system, *per se*. Mr. Wilgus.

However, as Mr. Murray has broached not only the question of the wisdom of the policy pursued by the New York Central, but also the wisdom of his own company's adoption of the alternating-current system north of Woodlawn, there is no recourse but to set forth the whole matter in sufficient detail for the drawing of correct conclusions. This is perhaps fortunate, for the present uncertainty in the minds of steam railroad officers is injurious to the advancement of the art of transportation, and peculiarly hurtful to the legitimate growth of electrification, whether by direct or alternating current.

There is really no quarrel between the alternating-current and direct-current systems. Both have legitimate fields. It is no more proper to compare them broadly than to contrast, say, a "Pacific" type passenger locomotive with a Mallet compound freight locomotive. They must be viewed in relation to a known service, and the care devolving on the engineer is to see that they are not misplaced. Legal restrictions, nature of traffic, mixture of steam and electric motive power, population, clearances, and other special conditions, all have bearings on the selection of the system of electrification best suited to any particular locality. It is a cause for congratulation that there is a choice of three systems, direct-current, alternating-current single-phase, and alternating-current three-phase, rather than but one system which would be unsuited to many localities seeking release from the limitations of steam.

The question, then, is whether or not either company has misapplied the system that it has adopted. The elements to be considered are:

(1) Physical and legal restrictions,
(2) Operating requirements,
(3) Safety,
(4) Reliability,
(5) Cost.

(1) *Physical and Legal Restrictions.*—The law taking effect July 1st, 1903, requiring the abandonment of steam in Park Avenue south of the Harlem River, within five years, gave an insufficient margin of time for the making of radical experiments, which, if unsuccessful, would cause delays alike distasteful to the public and the railroads. The temper of the public, atmospheric conditions in the Park Avenue Tunnel, and the congested nature of the Grand Central Terminal

Mr. Wilgus. yard operations, were such as to dictate the utmost speed in effecting the change, in the interests of safety and public comfort.

Of the two systems, direct current and alternating current, the former offered apparatus of proven reliability and efficiency, whereas the latter, at the time of the decision in the fall of 1903, was declared by its warmest advocates to be unsuitable for meeting the onerous conditions of the case. The state of the art was not sufficiently advanced to warrant the use of a system still untried in heavy trunk-line service; but, apart from this reason, there were others of even a more convincing nature.

The four-track Park Avenue Tunnel, 2 miles in length, with a head-room affording but 1 in. of clearance above the top of the rolling stock, is confined between the city street pavements immediately over the roof and the city sewers beneath. The overhead conductors and contact devices on the equipment of the alternating-current system would require at least 2 ft. 6 in. more head-room, obtainable only by radical changes in city sewers, to obtain consent for which would be very problematical; and the lowering of 8 miles of tracks in solid rock, in a smoke-laden tunnel through which flows at nearly all hours of the day a congested traffic of from four to five times the volume of the New Haven Company's traffic north of Woodlawn. Even if feasible and safe, the cost of doing this would be prohibitive. That such a change of the tunnel is impracticable from the legal standpoint is well shown by the fact that the public authorities stopped the company from drilling and blasting for electric ducts in the side-walls of the tunnel, during the few hours of the night when traffic conditions permitted the prosecution of work of even that simple character; and pipe ducts were substituted, hung from the tunnel walls. Then, too, the required method of rebuilding the Grand Central yard in sections during a period of many years, in conjunction with the construction of lofty buildings over tracks carrying traffic, prohibited the use of exposed overhead trolley wires alive with a current as dangerous as 11 000 volts.

That Mr. Murray can place these trunk-line conditions, in next to the largest city in the world, in the same class with the totally different and vastly less complicated problems on his own line, and at the Sarnia and Simplon Tunnels, is strange. The repeated promises and as frequent failures of Mr. Murray's company during the past year to complete its change to electricity, with the resultant serious delay to the Grand Central Terminal reconstruction, and annoyance to the public in the Park Avenue Tunnel, are the most speaking commentaries on the offer of Mr. Murray to undertake so blithely this task for others.

Apart from these physical objections to the overhead alternating-current system, there were serious legal obstacles.

The four-track Park Avenue Viaduct north of the tunnel, 1½ miles long, was built under legislative enactment that prescribed the exact

design. To modify this materially by the erection of trolley wires and supports would surely invite injunctions by abutting property owners, and resultant indefinite delays and enormous damages. The previous experience of the company with its neighbors in this thoroughfare, costing millions of dollars, taught a lesson that could not be disregarded. Mr. Wilgus.

The crowning legal obstacle was the objection of the city authorities to any form of overhead wires carrying high voltages along and over streets within the city limits. While the company has contended that transmission lines in the outer and sparsely-settled sections of the city, placed on the exterior edges of the right of way and passing far above the surface of intersecting streets, were permissible and even desirable, at least until the growth of population required a change to ducts, it did not feel that it could be denied that trolley wires carrying 11 000 volts immediately over and close to rolling stock and immediately beneath public travel on intersecting street bridges, would be sufficiently objectionable to invite ultimate adverse action by the public authorities that would entail a complete abandonment of a system costing the stockholders many million dollars.

With these absolute barriers to the use of high-voltage trolley wires, the New York Central, apart from other reasons, could not do otherwise than adopt the direct-current system, which in New York City is feasible and legal. The New Haven line, lying in the open country, did not have these obstacles, and adopted the alternating-current system.

(2) *Operating Requirements.*—The constantly increasing traffic in the congested Grand Central Terminal demanded a type of self-propelling electrical equipment that would minimize the number of switching movements across the throat of the yard. Experience elsewhere, also, had demonstrated the need of an elastic system of train operation, which, apart from the question of economies, would permit quicker acceleration, a more frequent service, and the regulation of the number of cars per train to the volume of traffic at different hours of the day. All these objects could be obtained by the use of multiple-unit cars, which, in the existing state of the art, seem best adapted to direct current.

The New Haven Company, in adopting alternating-current operation, rejected the use of the multiple-unit system, whereas the New York Central seized the opportunity to use it, with resultant immediate benefits to its operating department. That Mr. Murray's company now realizes its mistake is shown by its recent design of an alternating-current-direct-current multiple-unit train consisting of a motor car on each end of a six- or eight-car train. The success of such an arrangement is very questionable, from operating as well as electrical standpoints, and at least one large manufacturing company has declined to build it.

Mr. Wilgus. (3) *Safety.*—This item, referred to by Mr. Murray, should be considered in its twofold relation, to the employee and to the public, and not to one alone. Both, naturally, deserve the very best judgment and care in deciding upon the kind of distributing system to be used.

Experience has shown that the low-voltage third-rail, suitably protected, is not dangerous. During the period of a year and a half that the working conductors have been energized in the congested initial electric zone of the New York Central, not a fatality has occurred either to employees or the public, primarily due to the third-rail or transmission lines. Three instances have been due to trespassing on the transmission line, another to a porter reaching beneath the third-rail for a pack of cards, and one to a prior contributing cause.

On the other hand, up to the present time, the New Haven trolley-wire system and transmission lines have apparently caused thirteen fatalities, largely due to wires not being "beyond the reach of the tallest man."

Carelessness, no doubt, is responsible for the majority of these unfortunate occurrences, but is it not a duty to select the system which local conditions dictate as least dangerous to the negligent employee?

As to the public, the third-rail is entirely removed from neighboring thoroughfares, and the transmission lines in sparsely settled districts, spanning far above intersecting streets, seem to have the advantage of safety, superior to that of 11 000-volt trolley wires passing directly beneath street bridges of low clearance above the tracks and within a few inches of the passers-by.

On the direct-current system, in case of accident, the passenger is as well or better guarded from the third-rail by means of protecting sheathing and circuit-breakers, as he is from knocked down trolley wires which may affect not one but all four tracks.

May it not be concluded that, as measured by both practice and theory, the direct-current system, in the territory under discussion, is preferable to the alternating-current system, from the standpoint of safety, having in mind the local conditions?

(4) *Reliability.*—This question is of first importance to a trunk-line railroad. What has experience shown in the two systems under discussion?

During two representative months the delays per 1 000 locomotive-miles between Woodlawn and the Grand Central Station, due to locomotive failures, were as follows:

New York Central direct-current locomotives..	1.2	min.
Steam locomotives...........................	2.	"
New Haven alternating-current locomotives...	12.4	"

It will be noted that the alternating-current locomotives in this service caused eleven times as many train delays as the direct-current machines, and six times as many as due to steam power.

Since July, 1907, the New York Central has not had a single interruption of electrical service, whereas the New Haven Company, on its own territory north of Woodlawn, has had nine interruptions, of which four were very serious—in one instance lasting for 38 hours and necessitating a complete return to steam operation. Mr. Wilgus.

These facts demonstrate that, for reliability, the New York Central installation is far superior to that of the New Haven Company.

(5) *Cost.*—Comparisons of cost are absolutely valueless unless they are based on the same premises and conditions, and are in sufficient detail to permit analysis. To make a bare statement, unsupported by details, comparing the cost of the battery-less New Haven installation in the open country, with the one on the New York Central carrying four times the traffic through a section requiring expensive ducts instead of aerial lines, and into a terminal in the midst of a great city, is like showing side by side the cost per mile of the Union Pacific and Pennsylvania Railroads, without making allowance for the differences of topography, grades, number of tracks, terminals, and character of construction. Mr. Murray's ratio of 5 to 1 is misleading, as will be shown below.

Mr. Murray questions the accuracy of the saving of $300 000 per annum by the New York Central from avoiding the use of the alternating-current-direct-current locomotive, but he attempts to analyze the smaller part ($140 000) only, curiously enough not questioning the larger portion of the saving ($160 000 per annum) due to the requirement for a less number of locomotives. His silence on this point is most impressive if one considers the millions of dollars that have been spent by American railroads in grade reductions in order to reduce the number of locomotives for handling a given traffic—not to increase them.

Mr. Murray attempts to cast doubt on the statement of lower energy consumption for the direct-current locomotives by comparing the alleged actual results of the New Haven alternating-current locomotive on direct-current territory, with the theoretical assumptions of an arbitrator, whose decision, by the way, his company rejected.

Upon carefully checking the correct mileage, weights, and bills for current, he may also find that his watt-hours per ton-mile should be 50.7 instead of 41.9. However, in the following figures, the writer has used the figure which is most favorable to his contention.

The energy consumption upon which the item of $140 000 saving was based, was obtained in the following manner:

Several trial runs were made between the Grand Central Station and Woodlawn with both alternating-current and direct-current locomotives hauling identically the same weight of trains, at the same speeds, and with the same limited number of stops. The average results were:

Mr Wilgus.

Alternating-current locomotive (New Haven)	36.7	watt-hr. per ton-mile.
Direct-current locomotive (New York Central)	28.9	" " "
Saving in favor of New York Central locomotive, equal to 27%	7.8	" " "

Another comparison is available, as a check on the foregoing results:

New Haven: Average consumption south of Woodlawn, including more stops than were made in the foregoing trial runs	41.9	watt-hr. per ton-mile.
New York Central: Observations for direct-current locomotive, including more stops than were made in the foregoing trial runs	33.8	" " "
Saving in favor of New York Central locomotive, equal to 24%	8.1	" " "

To be well on the safe side, in showing a money saving, a difference of but 15% in favor of the New York Central locomotive is used in the following comparisons, this agreeing with a careful study of the characteristics of the several electrical parts of both locomotives:

New York Central Electric Zone saving, due to use of direct-current instead of alternating-current-direct-current locomotives:	
Direct-current locomotive annual requirements at the contact shoes, 36 000 000 kw-hr. at $2\frac{6}{10}$ cents	$936 000
Alternating-current-direct-current locomotive annual requirements at the contact shoes, 36 000 000 kw-hr. plus 15% excess, 5 400 000 kw-hr. = 41 400 000 kw-hr. at $2\frac{6}{10}$ cents	1 076 400
Annual saving	$140 400

New Haven loss, due to use of alternating-current-direct-current locomotives instead of direct-current locomotives south of Woodlawn:	
Direct-current locomotive annual requirements at the contact shoes, 10 500 000 kw-hr. at $2\frac{6}{10}$ cents	$273 000
Alternating-current-direct-current locomotive annual requirements at the contact shoes = 10 500 000 kw-hr. plus 15% excess, 1 575 000 kw-hr. = 12 075 000 kw-hr. at $2\frac{6}{10}$ cents	313 950
Annual loss	$40 950

It thus appears that by adopting a locomotive suited to the system over which it is to operate, the New York Central will effect a saving, for current only, over what would have to be expended if a locomotive had been adopted for operating on two systems, of $140 000 per annum; whereas the New Haven Company, by adopting the reverse policy, will suffer a loss between Woodlawn and the Grand Central Station of $40 950 per annum. Mr. Wilgus.

Mr. Murray asks why not compare the direct-current and alternating-current locomotives, apart from the complications attending the necessity of performing two functions by the latter. To do this requires a study of the first costs and annual costs of operation, by both systems, in the same territory, under precisely the same conditions, and embracing all variable elements. Therefore the writer has selected the New Haven Company's line, between Woodlawn and Stamford, having a total single-track mileage of more than 100 miles, in which is included a number of small yards.

The cost of the generating station for direct-current operation is found to be at least 20% cheaper than the one intended for alternating-current operation, for the reason that the generators for the latter, as built by the New Haven Company, are designed for three-phase output, but they are utilized for single-phase purposes, which largely cuts down their capacity. Then, too, the magnetizing of the motor fields of the alternating-current locomotives requires a large amount of wattless current not needed with the direct-current system. These conditions result in a much larger generator installation than would be needed for the direct-current system, and a corresponding higher cost.

To do the same work required on the 41 locomotives ordered by the New Haven Company, only 28 would be required for direct-current operation. This is due to the limit of five to six passenger cars to a single New Haven locomotive in order that the schedule speeds may be maintained, as compared with the ability of the New York Central type to make the same schedules with two or three times that number of cars. In other words, heavy trains require to be double- and triple-headed with the alternating-current locomotives, whereas but one direct-current locomotive of substantially the same weight and cost would suffice. The evil feature of this system is not only the heavier annual cost for repairs and for current needed by the additional alternating-current locomotives, but there is the serious operating handicap of holding spare units in readiness to attach to trains which, at the last moment before leaving the termini, are found to be heavier than was at first anticipated. The cost per locomotive of each type is taken at $30 000, but reliable information points to a considerably higher cost for the alternating-current locomotive, having approximately one-half to one-third the capacity of the direct-current locomotive.

The effect of this excess number of locomotives is to counteract the saving due to the superior efficiency of the distributing system of the

Mr. Wilgus. New Haven Company. For the 28 direct-current locomotives, it is estimated that 15 000 000 kw-hr. annually will be required at the contact shoes, based on which the requirements of the generating station bus-bars, with an efficiency of 81% (New York Central results) for the distributing system, would be about 18 500 000 kw-hr. The larger number of alternating-current locomotives for doing the same work increases the current demand at the contact shoes for the locomotive ton-mileage, so that, as compared with the 15 000 000 kw-hr. for direct-current operation, there is needed a 15% increase for the alternating-current system, or 17 250 000 kw-hr.

On the basis of 95% distributing-system efficiency, this makes the requirements at the bus-bars of the generating station 18 200 000 kw-hr. Thus it is seen that, for the two systems applied to the territory in question, the demands of current at the generating station bus-bars are nearly alike.

The depreciation of the alternating-current system under New Haven conditions is greater than that of the direct-current system, because of the comparatively short life of the trolley wires and catenary construction, which are subjected to abrasion and corrosion. This item is aggravated in this instance by the combined operation of electricity and steam. In both systems, due to deterioration and obsolescence, a life of twenty years may be used, except for the trolley wires and catenary system, for which an extreme life of five years is assumed. Personal observation prompts the belief that the last named period is much greater than will be actually experienced.

Maintenance of generating plants and distributing systems is taken at 2% per annum for both systems. Observations of the annual costs of maintenance of third-rail and overhead work show that they are about equal where the conditions are similar, provided that separate provision is made for the more rapid depreciation of the latter.

The maintenance of the New Haven locomotives is assumed to be 6 cents per locomotive-mile, as compared with 3 cents per locomotive-mile for the New York Central locomotive. The excessive number of locomotive failures, the complication of parts, and the known large size of repair gangs at Stamford and at the Grand Central Terminal indicate that the cost of maintenance of the New Haven alternating-current locomotives is much in excess of the assumed figure. Thirty thousand miles per annum per locomotive is assumed for both systems.

The actual experience of the New Haven Company with injury to employees due to contact with 11 000-volt trolley wires warrants the assumption that payments for damages will be at least five times those that are chargeable to the use of the third-rail of the direct-current system. Low clearances at street bridges for trainmen and other employees; possibilities of falling wires caused by abrasion and corrosion and from high winds similar to those that recently felled neighboring wires, trees, and poles, causing a temporary suspension of New Haven

service; and proximity of signals to the trolley wires—all these aggravate the New Haven situation. Mr. Wilgus.

With these explanations, the following statements have been prepared.

COMPARATIVE ESTIMATED COSTS OF ELECTRIFICATION OF THE N. Y., N. H. & H. R. R. FROM WOODLAWN TO STAMFORD.

Items.	Direct-current.			Alternating-current.		
Generating stations.....	....		$1 300 000	....		$1 500 000
Distributing systems....	Sub-stations.........	$600 000		Trolley system.	$500 000	
	Working conductors, etc..................	800 000		Overhead bridges, etc..	750 000	1 250 000
	Transmission lines, etc..................	550 000	1 950 000			
Rolling stock.	28 locomotives................		850 000	41 locomotives..........		1 250 000
Totals......	....		$4 100 000*	....		$4 000 000†

* Based on a liberal estimate of probable cost.
† Based on a liberal estimate of probable cost. The New Haven Company's annual reports indicate a higher cost.

COMPARATIVE ESTIMATED ANNUAL COSTS OF OPERATION BY THE DIRECT-CURRENT AND ALTERNATING-CURRENT SYSTEMS: WOODLAWN TO STAMFORD.

Items.	Direct-current.			Alternating-current.		
Fixed Charges { Int. 4%, Taxes 1½%, Ins. and Risks 1½% }	7% on $4 100 000.....		$287 000	7% on $4 000 000......		$280 000
Depreciation. (Annual sums required to accumulate a fund sufficient to extinguish cost at expiration of assumed life)................	Assumed life of 20 years = 33.60 per $1 000............		137 760	Assumed life of 20 years, except for the trolley system = 33.60 per $1 000......	$117 600	
				Assumed life of trolley system, 5 years = 184.60 per $1 000......	92 300	209 900
Maintenance.						
Generating stations	2% of cost....	$26 000		2% of cost...	$30 000	
Distributing system.............	" " "	39 000		" " " ...	25 000	
Rolling stock.....................	840 000 miles @ 3c.......	25 200	90 200	1 230 000 miles @ 6c.......	73 800	128 800
Operation (exclusive of maintenance).						
Generating stations	18 500 000 kw-hr. @ $\frac{5}{10}$ cent	$92 500		18 200 000 kw-hr. @ $\frac{5}{10}$ cent........	$91 000	
Sub-stations.....................	3 @ $6 000...	18 000				
Distributing system............	Personal injuries......	5 000	115 500	Personal injuries......	25 000	116 000
Grand totals.................	....		$630 460	....		$734 700

Mr. Wilgus.

Excess cost of alternating-current over direct-current north of Woodlawn	$104 240	per annum	= 16 per cent.
Add excess cost of operating alternating-current locomotives on direct-current territory....	40 950	" "	
Total excess cost to New Haven Company by adoption of alternating-current system	$145 190	" "	= 23 per cent.

It will thus be seen that, under the conditions of traffic on the New Haven Road, the alternating-current system costs 16% more to operate than the direct-current system, and if the excess cost of operating alternating-current locomotives on direct-current territory is considered, the loss is 23 per cent. If, in the estimate, consideration had been given to the use of the multiple-unit cars for suburban service, instead of locomotives, the direct-current system would show even a greater saving.

Summarizing all the cost figures for steam, direct-current and alternating-current service, on both the New York Central and the New Haven lines, and adding locomotive wages, so as to agree with the conclusions given in the paper, the comparative annual results are:

New York Central (Ultimate Electric Zone).

Steam.................. 1 000 000 thousand car ton-miles at $2.77	$2 770 000
Electric: direct-current.. 1 000 000 thousand car ton-miles at $2.02	2 020 000
Difference: Saving.............................	$750 000

New Haven Company (Woodlawn to Stamford).

	Direct-current. (Not adopted.)		Alternating-current. (Adopted.)		Loss by alternating-current system.
Steam	262 500 thousand car ton-miles @ $2.77	$727 125	262 500 thousand car ton-miles @ $2.77.........	$727 125	
Electric......	262 500 thousand car ton-miles @ $2.71 ±.......	711 310	262 500 thousand car ton-miles @ $3.11 ±.......	815 550	
Difference...	Saving	$15 815	Loss.....................	$88 425	$104 240
			Add for loss south of Woodlawn	40 950	40 950
			Total loss...............	$129 375	$145 190

These figures are not claimed to be absolutely accurate, as the New Haven Company's detailed costs have been withheld, but they are based on careful estimates made on the ground, and are believed to be sufficiently accurate to enable one to form the reasonably correct conclusion that, under the conditions existing on the territory in question, the New York Central, by reason of its adoption of the direct-current system, will ultimately show a very large saving over steam, say, approximately, $750 000 annually; and that the New Haven Company, instead of showing the saving over steam of, say, $15 000 annually, which might have reasonably been expected from the use of the direct-current system, will show a loss of, say, approximately, $129 375 annually, a total net loss of, say, $145 190 annually. Mr. Wilgus.

Conclusions.—From the foregoing it seems proper to draw the conclusions that the early decision of the New York Central to adopt direct-current to fit its local conditions, has resulted in the following advantages that would have been denied had the system been selected that Mr. Murray's company later adopted:

(a) Timely relief of the public from the products of combustion and noise of the New York Central's steam locomotives in the Park Avenue Tunnel and along Park Avenue;

(b) A reliable and efficient substitute for steam service;

(c) Compliance with the law, and hence absence of danger of drastic fines for use of steam locomotives in Park Avenue after July 1st, 1908;

(d) Promise of substantial economies of operation which, upon the completion of the change to Harmon and North White Plains, are believed will approximate $750 000 annually.

As previously stated, Mr. Murray misinterprets the intention of the paragraph he first quoted; and, in "controverting the statements made against the alternating-current system," he has undertaken an unnecessary task. The writer has genuine pleasure in cordially agreeing with him "that the near future will see the high-voltage distribution system, with its attendant alternating-current locomotive, propelling trains from and between terminals * * * "; but, for the sake of both stockholders and public, let it not be forgotten that there are other items, in addition to "density of traffic," which, if ignored, lead to a misapplication of a worthy system.

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TRANSACTIONS

Paper No. 1080

SAFE STRESSES IN STEEL COLUMNS.*

BY J. R. WORCESTER, M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. HENRY B. SEAMAN, LUZERNE S. COWLES, CHARLES M. EMMONS, HENRY S. PRICHARD, HORACE E. HORTON, F. P. SHEARWOOD, LEWIS D. RIGHTS, A. W. CARPENTER, C. J. TILDEN, ERNST F. JONSON, R. D. COOMBS, WILLIAM CAIN, AND J. R. WORCESTER.

The subject of a proper allowance for stresses in columns has been treated so often by theorists that it may seem as though no more could be said on the subject without danger of exhausting the patience of the engineering profession; but, in spite of all the theories, the practice of steel designers, as shown by the specifications in general use to-day, may well bear further consideration.

The reason for this is that all "rational" column formulas, based on the elastic properties of the steel, are founded on considerations which are applicable only to ratios of length to radius of gyration far beyond those allowed in actual construction. It is known, in a general way, that steel in compression should not be strained as high as in tension, and there is a popular impression that the only reason for this is that when the ratio of l to r increases above 0, or, at most, above a value very little above 0, the strength becomes lessened rapidly on this account; but there has been a growing tendency to neglect the fact that, even in very short columns, there is not the same unit strength manifested against compressive and tensile stresses.

* Presented at the meeting of February 19th, 1908.

The reason for this difference is manifest from a moment's consideration. It may be admitted that, within the elastic limit, the modulus of elasticity is practically the same, whichever way the metal is strained—in tension or compression. One may even go further and admit that the elastic limit is practically the same for both stresses; but, what happens after the elastic limit is passed? In tension, the member merely straightens out—if it is not straight to start with—and stretches, while with every increase in length comes an increase in resisting strength until the ultimate strength is reached, the final strength being nearly twice as great as it was at the elastic limit.

In compression, however, soon after the elastic limit is passed, the column will cripple, and the more it cripples the weaker it becomes. It is not necessary to consider the ideal conditions of exact equilibrium in the resisting power of a section, when crippling would not take place, because the equilibrium—unstable at the best—is not attainable in practice. On the other hand, it must be admitted that the ductility, which is of such great advantage in tension, is not present to an appreciable amount in compression, and that the ratio between working stresses and destructive stresses in all structures depends on the compression members, and not the tension, when anything like equal working units are allowed in the two.

An examination of the results of tests of full-sized columns made by Tetmajer, Marshall, Christie, Bouscaren, Strobel, Lanza, and the Watertown Arsenal, shows strengths of wrought-iron columns, in which the $l \div r$ does not exceed 120, of from 16 000 to 43 000 lb. per sq. in., and for mild steel, from 18 000 to 46 000 lb. per sq. in. By far the larger part of these range between 22 000 and 34 000 lb. for iron, and between 22 000 and 46 000 lb. for steel. It is very noticeable, also, that one finds results of more than 28 000 lb. with the longest length and less than this amount with values of $l \div r$ as small as 30. While the axis drawn through the central portion of the group of these experiments, when plotted, shows some inclination toward lower values for increased length, the center of the group lies at about 30 000 lb. when $l \div r = 90$, and there is very little increase in strength manifested in the tests with a lesser length than this. It is apparent, therefore, that if the compression is allowed to run as high as 16 000, the factor between working stress and ultimate will not exceed 2. In tension members, on the other hand, the corresponding factor is nearly 4.

The answer to this argument is, of course, that nobody cares what the factor between working strain and ultimate may be, as one is really interested only in the elastic limit, which it is never intended to reach. Is it not time to call a halt on this line of reasoning? Have not engineers been overconfident in their ability to design structures so that all possible contingencies are taken into account? One would not willingly make use of a material in tension which had no stretch beyond the elastic limit, yet it would be in no way more hazardous than to neglect the fact that such is the case with compression members, unless a greater factor below the elastic limit were allowed in these.

The history of the development of the column formulas used in bridge specifications may shed a little light on the way in which unit strains have crept up.

The adaptation of the Gordon formula to wrought-iron columns by Rankine had for a numerator 36 000 lb. That is, Rankine recommended that this value be assumed for the ultimate strength of wrought iron in compression of short columns. The earlier specifications for railroad bridges, in which Rankine's formula was used, recommended 7 500 or 8 000 in the numerator when 10 000 was used for tension, and this difference between the numerator of the compression formula and the tensile unit has been retained to a large extent in specifications until recently.

When the straight-line formula was first introduced, it was recommended for the reason that a straight line could be drawn that would coincide very well with the plotted results of experiments for ratios of l to r between 90 and 150, and that, in giving less values than experiments warranted above this point, it erred on the side of safety. The straight line, thus drawn, when prolonged the other way, reached the $l \div r = 0$ line at about the tensile value of the steel, making the formula take the form, $A = B - C \times \frac{l}{r}$, in which A = the allowable compressive stress, and B = the allowable tensile stress. This simple form appealed strongly to engineers, and was readily accepted by many, but the fact was not recognized by all that the line when plotted goes far above the experiments for values of $l \div r$ less than 90. The tables by C. L. Strobel, M. Am. Soc. C. E., for the strength of Z-bar columns were based on a straight-line formula, but this is a notable instance of recognition of the error of the formula for short lengths,

because he limited his stresses to 12 000 when the straight line went higher than this amount.

At this point, may be noted what seems to have been an unwarranted change in specifications, due to the reprehensible practice of copying from one to another with slight changes. There have always been many engineers who liked the form of the Rankine formula and refused to give it up. Many appear to have been struck with the simplicity of the straight-line formula in having the unreduced compression unit the same as the tension, and, wishing to take advantage of this feature, but still adhering to the Rankine form, they adopted the tension unit for the numerator of the formula. This throws the curve entirely above the field of tests, and, apparently, cannot be defended by any reasoning.

A later development of the specifications which are based on the form of the Rankine formula and still retain the tension unit in the numerator, is to adopt a lower constant in the denominator. This, by some, is made 20 000, and by others, 8 000. The former brings the curve within the outer limits of the group of tests, while the latter passes well through the middle of the group for values of $l \div r$ greater than 50, but is above the group for lower values.

Perhaps enough has been said to show that the formulas in general use to-day need to be sawed off at the end toward low values of $l \div r$. It may also be said that they all need to be amputated at the other end. Mr. Schneider, years ago, suggested that values of $l \div r$ greater than 100 should not be allowed in main members, and this limitation, with slight variations, has been generally accepted since that time as an essential of good practice.

If, then, the Rankine formula be used, with the numerator value equal to the tension, and the compression stress be limited to, say, 75% of the tension, and the value of $l \div r$ to 100, or thereabouts, one obtains for a diagram a horizontal line running to a cusp, then a concave curve running to another cusp, then another straight line. Could anything be more irrational? The straight-line formula is little better; the only difference being, that, in the middle portion, there is a straight line instead of the curve. How much better it would be to use a continuous curve throughout, embodying its own limitations at each end!

The late J. B. Johnson, M. Am. Soc. C. E., suggested this same

thought in his book on Modern Framed Structures, and proposed a parabola. This is safe and simple; though, if the vertex is kept down to a safe value of stress for short lengths, and the limitation of $l \div r$ is made not higher than 120, the central portion of the curve does not reach as high as tests would warrant.

An elliptical curve fits the case much better, the ellipse being drawn with its center at the zero value for both stress and $l \div r$, and having for one semi-diameter the limiting value of $l \div r$, and for the other the limiting stress for zero lengths. The form of this equation is:

$$A = B\sqrt{1 - \frac{l^2}{(C\,r)^2}}$$

in which A = the allowable stress, B = the maximum stress at $l \div r = 0$, and C = the maximum value of $l \div r$ allowed. This curve is easy enough to plot as an ellipse, but, if a diagram be only arranged so that on the scale of ordinates B is of the same length as C on the scale of abscissas, the curve becomes the quadrant of a circle.

The diagram, Fig. 1, illustrates graphically a number of curves of well-known specifications, together with the results of tests, by the experimenters previously referred to, reduced so as to allow for a proper factor of safety. This reduction is made so that the experimental results can be compared with the formulas in their usual forms. The reduction applied is proportional to the ratio between the tension unit and the ultimate tensile strength of the metal. That is, for wrought iron, the test values are multiplied by $\frac{16\,000}{50\,000}$, and for steel the multiplier is $\frac{16\,000}{60\,000}$.

The proposed formula, as plotted, is based on limiting values of compressive stress of 12 000, and of $l \div r$ of 120, which appear to be warranted by experiments and by good practice, and, as the scales are arranged, the curve is circular.

The writer puts forward a new formula with great diffidence, knowing well that custom is a very difficult thing with which to contend, and how cold a reception new compression formulas have met in the past; but, considering how poorly the formulas now in use fulfill the requirements, and realizing that the public is fully awakened at the present time to their insufficiency, the time seems to be opportune for at least suggesting the possibility of an improvement.

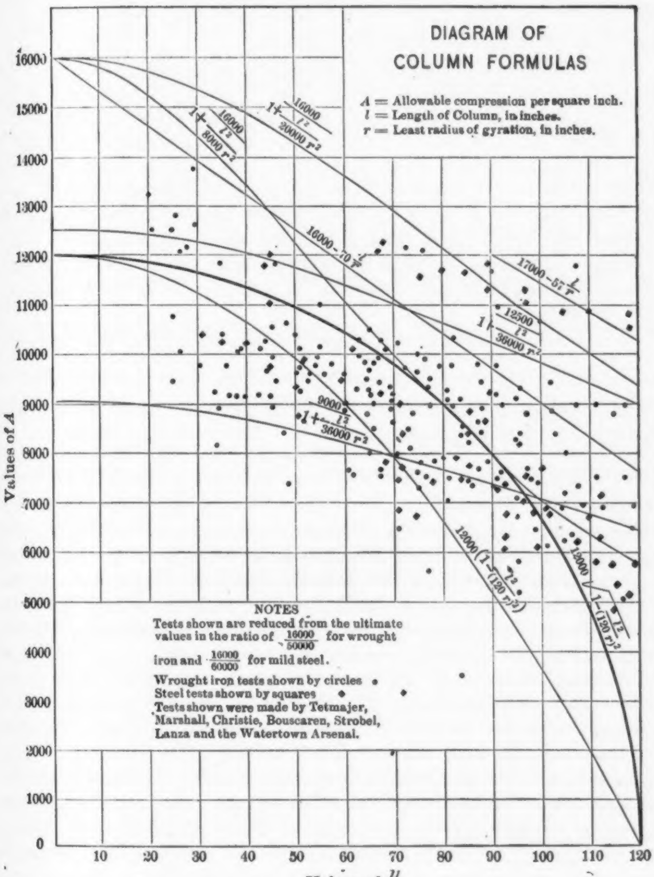
DIAGRAM OF
COLUMN FORMULAS
A = Allowable compression per square inch.
l = Length of Column, in inches.
r = Least radius of gyration, in inches.
NOTES
Tests shown are reduced from the ultimate values in the ratio of $\frac{16000}{50000}$ for wrought iron and $\frac{16000}{60000}$ for mild steel.
Wrought iron tests shown by circles
Steel tests shown by squares
Tests shown were made by Tetmajer, Marshall, Christie, Bouscaren, Strobel, Lanza and the Watertown Arsenal.
Values of A
Values of $\frac{l}{r}$

FIG. 1.

DISCUSSION.

Mr. Seaman. HENRY B. SEAMAN, M. AM. SOC. C. E. (by letter).—It may be too early yet for a Special Committee to advise as to the proper column formula to be used in structural work, but Mr. Worcester's paper brings us one step nearer its appointment.

To the writer's mind, there never has been sufficient reason for abandoning the Rankine formula. The basis of its formation is the provision that a column receives both direct strain and bending strain. The direct strain is readily provided for, and the effect of bending is found by experiment, the results of which are used in determining the coefficient of r^2. It would seem better to plot the results of these tests upon the basis of ultimate strength, rather than working strength, as it keeps the mind more directly on the actual data observed. The formula can then be modified for working strength, either by taking a certain proportion of the numerator as a factor, or by other modification, if preferred.

It should be remembered that details are designed upon an assumed value of $\frac{l}{d} = 12$, where l equals the length and d the least diameter of a solid rectangular column. In designing columns, therefore, a greater strain should not be permitted than that for which the details are designed, that is, for a less value of $\frac{l}{d}$ than 12. This serves, as Mr. Worcester has expressed it, to truncate the formula for very short columns. Since the Rankine formula, however, provides for bending as well as compression, it conforms with the tests on long columns better than does the straight-line formula, and, for that reason, to the writer's mind, is the most valuable formula we have. There has never seemed to be any excuse for the adoption of a straight-line formula, except simplicity in plotting and ease in memorizing. Confessedly, it does not conform to tests on long columns, it is not applicable beyond certain restricted limits, and finally, since it involves r instead of r^2, it cannot be used as readily without the assistance of tables; yet tables will assist equally well with any formula.

A recent study of the tests mentioned by Mr. Worcester has led the writer to adopt the following formulas:

For Steel and Wrought Iron:

$$p = \frac{a}{1 + \frac{l^2}{8\,000\,r^2}}$$

For Cast Iron:

$$p = \frac{a}{1 + \frac{l^2}{1\,000\,r^2}}$$

Mr. Worcester very properly calls attention to the fact that the failure of a column occurs when it begins to cripple, while, with the tension member, if allowed time to rest, the material becomes even

stronger because of the work of overstrain which it has received. This would enable us to permit a higher factor of safety upon tension members than upon compression members, were it not for the fact that a permanent elongation of a tension member would deform the structure to such an extent as to change the strains for which it was designed, and possibly cause failure on that account. It must also be remembered that the element of fatigue—and possibly that of momentary impact—need not be considered in the bending of the column, and therefore the extra material used, in order to prevent bending, is an additional factor of safety, which the tension member does not possess. Mr. Seaman.

The recent tendency in structural design seems to be to increase the live loading by a given factor in order to derive an equivalent static strain, and then to design the parts for these static strains, rather than the old method of using a factor of safety to cover defects in material, increase, and extraordinary effects of loading, etc. If the live-load strains can be increased so as to cover all possible contingencies, and if a dead load can be assumed which will not be exceeded under any circumstances, it would seem safe to place the allowable strain at one-half or two-thirds of the elastic limit. It is on this basis that long-span bridges are designed; and, by the adoption of a formula in which this factor would vary with the various lengths of span, the same method of proportioning could be adopted for shorter spans. Future specifications will probably tend in the direction of some such method of design.

Luzerne S. Cowles, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Worcester need hardly offer an apology for continuing the agitation concerning compressive stresses to be allowed in designing structural steelwork. The writer, from the beginning of his career, and in fact during his college course, has been decidedly baffled by the numerous formulas for allowable safe compressive stresses, and had begun to believe that most so-called "rational" formulas were made up to assist the designer in making a comparatively safe guess. The question arises, however, as to whether all the commonly accepted formulas do really give the margin of safety that is desired and is assumed to exist. Mr. Cowles.

C. C. Schneider, Past-President, Am. Soc. C. E., has frequently called attention to the fact that the elastic limit, and not the ultimate strength, should be especially considered in deciding the real factor of safety. This gives, for tension, and supposedly for compression, a real factor of approximately 2, on the basis of 16 000 lb. per sq. in. for static loads. The writer agrees with this, particularly where compression is involved.

In the light of recently published data of experiments on full-sized compression members, it would seem that this real factor of safety of 2 had even been seriously encroached upon, leaving far too lean a

Mr. Cowles. margin of safety for structures where human life is at stake. When one considers the astounding results of Mr. Buchanan's tests,* where the fiber stress at crippling, even for so-called "short" columns, was below the accepted elastic limit, it seems to be high time to consider reducing the allowable unit stress for compression below that for tension, even though the modulus of elasticity and the elastic limit appear in the laboratory, and no doubt are, approximately the same for each.

Most railroad bridge specifications insist that no compression member shall have a length exceeding 100 times its least radius of gyration, except for bracing, where a ratio of 120 may be used. In other words, a main compression member in which the $\frac{l}{r}$ is 100, will carry safely 9 000 lb. per sq. in., whereas the use of a main member in which the $\frac{l}{r}$ is greater than 100 is disapproved. This is according to a standard straight-line formula, and it seems that the use of very "long" columns is not discouraged to the extent that it should be.

Is not then the really "rational" formula one which gives comparatively low results for the allowable fiber stress for the longest columns consistent with good design, and errs on the side of safety for the occasional exceptionally short strut? Mr. Worcester's proposed formula seems to fill these conditions, and while it may not be perfect in its present form, it is surely a step in the right direction, and furnishes a basis for a truly sensible formula. With his customary modesty, the author of the proposed formula has failed to point out its commendable features. The writer suggests the following:

(a).—Reducing the allowable stress for "short" columns so as to give a reasonable factor of safety;

(b).—Discouraging the use of columns in which the $\frac{l}{r}$ is greater than, say, 90 to 100;

(c).—Placing the allowable stresses, for columns in which the ratios of $\frac{l}{r}$ lie between 30 and 90, at figures which are slightly below the average results, as shown by numerous tests.

Mr. Emmons. CHARLES M. EMMONS, M. AM. SOC. C. E. (by letter).—The writer is much interested in this paper. Mr. Worcester's plotting of the results of actual tests, reduced by a safe working factor, and also his plotting of the several column formulas, to the same scale, reveals very graphically the inconsistencies and the wide divergencies of these formulas.

The writer is not as fully impressed with the idea of a formula being self-limiting at the highest allowed value of $\frac{l}{r}$. In attempting

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Mr. Emmons.

to do that, the author's curve appears to be as inconsistent with the tests as would be a straight-line formula. The writer realizes that a formula should be of such form that, if the allowed value of $\frac{l}{r}$ be not fixed arbitrarily, it will yet be in no wise dangerous, for the reason that someone with more "nerve" than judgment, or through ignorance or other cause, will occasionally use such a formula as $\dfrac{12\,500}{1 + \dfrac{l^2}{36\,000\,r^2}}$ to the limit. This danger was just lately brought to the writer's attention in a case where, for compression members, more than 10 ft. long, having a stress of about 4 000 lb., a prominent engineer used a single angle $2\frac{1}{2}$ by 2 by $\frac{1}{4}$ in. The $\frac{l}{r}$ is more than 300, and yet, according to the formula, it should carry the load safely.

The use of any formula which may be based on a series of observations, like those given by the author, should not, with confidence, be pushed very far beyond the limits of those observations. Such a formula, however, should take full advantage of what is indicated as safe by those observations.

Again, the formula proposed by the author, where one would be practically limited to $\frac{l}{r} = 112$, would be prohibitive, in many classes of work, especially for secondary members.

In view of these considerations, the writer would prefer the use of a parabola with values, say, $12\,000\left(1 - \dfrac{l^2}{(160r)^2}\right)$, with the limiting allowed value of $\frac{l}{r}$ prescribed. The formula is practically self-limiting at $\frac{l}{r} = 150$. It will be observed that this curve gives practically the same values as the circle up to $\frac{l}{r} = 80$, and from that point it follows the tests far better, taking advantage of what the tests indicate as safe, and yet in no case becoming dangerous.

Mr. Prichard.

HENRY S. PRICHARD, M. AM. SOC. C. E. (by letter).—The introductory paragraphs of this paper give the impression that the author attaches slight importance to theory in regard to columns. Would it not be well to discriminate somewhat in this regard? It is unfortunate that a single word, "Theory," is popularly used (with the sanction of the dictionaries) to designate "a body of the fundamental principles underlying any science or application of a science," and the radically different conception "a proposed explanation designated to account for any phenomena," no matter how visionary the assumptions, fallacious the argument, or foolish the conclusion. It is natural and proper that

Mr. Prichard. many of the proposed explanations of the behavior of columns should be held in light esteem, but it is highly desirable that engineers should understand and apply the principles of mechanics to the design of columns. Without such an understanding, the phenomena observed in practice and in the numerous compression tests are to a considerable extent a set of seemingly discordant facts.

Referring to the fact that the practice of steel designers with regard to columns may well bear further consideration, the author states:

"The reason for this is that all 'rational' column formulas, based on the elastic properties of steel, are founded on considerations which are applicable only to ratios of length to radius of gyration far beyond those allowed in actual construction."

It is difficult to reconcile this statement with the analyses and equations developed by Euler, Cain, Fidler, Marston, J. B. Johnson, Moncrieff, and others who have determined important facts regarding short as well as long columns by reasoning based on the elastic properties of steel and iron. The names of Tredgold, Gordon, and Rankine have purposely been omitted from this list for the reason that the formula which they, by successive steps, developed is based on the erroneous application to columns of the principle, strictly applicable to beams, that the greatest possible deflections within the elastic limit, of beams similar as to section, manner of loading, and end conditions, are proportional to the squares of their lengths multiplied by the elastic limit. In columns, under analogous conditions, the greatest deflections within the elastic limit are proportional to the squares of their lengths, multiplied, not by the elastic limit, but by the differences between the elastic limit and the mean compressive stresses in the various columns.

Euler's formula applies only to long columns, but he should be included among those who, by analysis, have determined facts as to short columns, for the reason that his formula carries with it the necessary consequence that, under ideal conditions, columns which are too short to have Euler's formula apply to them will have a uniform distribution of stress and no deflection, up to the elastic limit, a condition which is sometimes closely approached in laboratory tests. In practice, of course, the conditions may be far from ideal, but Fidler, Marston, J. B. Johnson, Moncrieff, and others have made valuable and instructive analyses of the effect, within the elastic limit, of departures from ideal conditions.

The author objects to the practice of using the elastic limit as the criterion of strength without regard to the ultimate. When rest occurs between the periods of straining beyond the yield point, the elastic limit, which at first is somewhat below the yield point, can be raised somewhat above it, thus making a permanent gain in strength, the usefulness of which is greatly lessened by the fact that when

structural steel of the usual quality is overstrained it becomes very ductile. Mr. Prichard.

When only a small portion of a steel member is overstrained, and the conditions are such that a very small flow of the ductile metal brings relief, the overstrained steel, by regaining its elasticity during a rest, accommodates itself to the conditions with comparatively slight distortion. Thus ductility, combined with the recuperative powers of the steel, may be useful in adjusting the length and shape of members and details, and in raising the strength of pins, etc., but if the stress over the entire cross-section of a member is even slightly greater than the yield point, and there is no other direct path for it to follow, the member, if in compression, will buckle, unless it is very short and stiff, and, if in tension, will elongate so much that it will not only be irreparably injured, but will cause ruinous distortion in the remainder of the structure, and possibly the failure of some adjacent compression member, to the supposed weakness of which the disaster may be erroneously attributed.

Between ruinous distortion and collapse there is a great difference: Ruinous distortion means the loss of the structure, while collapse may, in addition, cause great damage and loss of life. The possession of strength in excess of the yield point, even though it be but temporary, is, therefore, of some value, and a somewhat higher unit stress could be allowed in members which possess it than in those which do not. It should be remembered, however, that even a tension member, from some action which starts from a nick, a flaw, a jagged edge, a thread, or a rivet hole, or in some detail from no distinctive point, may break without marked elongation, especially under shock.

There is no analytical method by which strength within the elastic limit can be equated with strength beyond it, but, other things being equal, would not an advantage of 4 000 lb. per sq. in. fully offset the absence of any strength there may be in a highly ductile metal beyond that limit? To assist in considering this question, the elongations beyond the yield point during a test of a fairly typical eye-bar are submitted in Table 1.

TABLE 1.—Elongation of a Typical Steel Eye-bar, Measured in a Length of 262½ In. from Center to Center of Pins.

Load per square inch, in pounds.	Elongations:		Load per square inch, in pounds.	Elongations:	
	Inches.	Percentage.		Inches.	Percentage.
35 000	0.55	0.02	45 000	6.92	2.64
36 000	2.07	0.79	50 000	9.75	3.72
37 000	2.75	1.05	55 000	13.60	5.17
38 000	3.77	1.44	60 000	20.63	7.84
39 000	4.21	1.60	64 410	38.25	14.6
40 000	4.56	1.74	56 710	40.65	15.5

Mr. Prichard. A strain of 4 500 lb. per sq. in. in excess of the yield point, with a percentage of elongation equal to the percentage in the bar cited, if it occurred in the diagonal eye-bars of a bridge panel 25 ft. long by 25 ft. high, would cause a distortion of the bridge, in one panel length, of 9½ in. This would be a severe test of the floor system and top chords.

The relation of the yield points in compression to those in tension was well shown by a set of comparative tests by the late Charles A. Marshall,* M. Am. Soc. C. E., a synopsis of which is given in Table 2.

TABLE 2.—COMPARATIVE TESTS, IN TENSION AND COMPRESSION.

All from the same blow of Bessemer steel as it came from the rolls.

Size and shape of test piece.	YIELD POINT.			Ultimate strength in tension, in pounds per square inch.
	Compression, in pounds per square inch.	Tension:		
		In pounds per square inch.	Percentage of ultimate.	
4 by ½ in.	Not given.	53 800	75.5	71 255
$\frac{8}{16}$ in. square	49 055	47 815	68.8	69 390
3 by ⅜ in.	Not given.	47 363	68.9	68 657
¾ in. round	47 300	46 090	66.8	68 995
3 by ⅛ in.	Not given.	44 417	65.8	67 527
¾ in. square	43 845	44 273	64.7	68 427
1 in. round	46 020	44 202	65.0	67 970
1 in. square	Not given.	43 560	63.6	68 510
4 by ¾ in.	" "	41 527	62.1	66 917
3 by ¾ in.	" "	41 447	61.9	66 987
4 by 1 in.	" "	41 415	62.4	66 230
1¼ in. square	42 300	41 060	60.4	67 973
1¼ in. round	43 460	40 747	60.8	67 040
1⅜ in. "	41 290	40 275	60.7	66 363
1¾ in. "	42 075	40 017	60.3	66 333
3 by 1 in.	Not given.	39 397	59.1	66 700
1⅜ in. square	42 740	39 317	58.8	66 833
3 by 2 in.	Not given.	39 302	59.1	66 537
3 by 1¼ in.	" "	38 482	57.7	66 640
2¼ in. square	39 940	38 810	....	Not given.
2 in. round	38 830	38 207	58.2	65 663
1¾ in. square	40 630	38 193	57.5	66 400
3 by 1½ in.	Not given.	37 820	57.0	66 342
4 by 1½ in.	" "	37 580	....	Not given.
2¼ in. round	" "	37 000	56.5	65 460
4 by 1¼ in.	" "	36 680	....	Not given.
2⅜ in. round	36 840	36 100	....	" "
3 by 1¾ in.	Not given.	35 917	54.6	65 762

In Table 2 the strength of the steel increases, in a general way, as the size and thickness of the sections are reduced. A similar variation in the strength of wrought iron was shown, by tests made by the United States Board on Testing Iron and Steel,† to be due to reduction in rolling. In most cases, the results for compression are each an average of two tests, and for tension, of three or four tests. The average of the yield points given in Table 2 for compression is 1 432 lb. per sq. in. greater than for the corresponding results for tension. The results

* *Transactions*, Am. Soc. C. E., Vol. XVII, p. 68.

† Vol. I, 1881, pp. 35-45.

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of these tests cannot be applied directly to tension in eye-bars. The fact that eye-bars are annealed puts them in a different class from material as it comes from the rolls, as the steel is softened, some of the good effects of rolling are taken away, and the proportion of yield point to ultimate is lowered, especially if the bars are cooled slowly. Except in rare cases, steel as it comes from the rolls will have a yield point in tension exceeding 55% of its ultimate strength, while the average of all the tests (some 570 or more) of full-sized eye-bars, made during the last few years at the Ambridge plant of the American Bridge Company, gives a yield point equal to 52½% of the ultimate strength of the full-sized bar, with variations above and below this percentage. It is not wise to count on a yield point of more than 50% of the ultimate strength of the bar.

It appears from the foregoing that the higher yield point in compression, of steel as it comes from the rolls, as compared with the yield point of annealed eye-bars, would about offset the advantage which the latter possesses of some temporary strength in excess of the yield point, even when the same ultimate tensile strength is specified for the eye-bars, as determined by full-sized tests, and steel for compression members, as determined by specimen tests. A comparison between the strength of steel in compression and the tensile strength of built steel members is a different matter, and introduces a somewhat different set of questions. Few engineers, however, unless they are opposed to pin connections, would permit a higher unit stress for the net area of a built tension member than for an eye-bar.

With the exception of columns made of pipes and single angles, practically all columns are built of sections and plates. The rivets are usually assumed to fill the holes and take the place, as far as compression is concerned, of the sections they replace. As a matter of fact, they do not completely fill the holes, and it is very doubtful whether they wholly make good the loss in section. They are seldom placed closer than an average of 4 in., and probably they are more than half as effective as the metal they replace. On this basis, the allowed stress per unit of gross section of column area would be about seven-eighths of the allowed stress per unit of net section in tension; that is, if 16 000 lb. per sq. in. is allowed in tension, 14 000 lb. would be a corresponding limit for compression. There are other considerations, however, chief of which is the weakening influence of slenderness in either the column as a whole, or in its details.

Notwithstanding the large number of tests that have been made in the endeavor to determine the influence of slenderness (Moncrieff, in his paper on "The Practical Column," cites more than 1 000),* the practice of engineers in this regard, as shown by the author, is very diverse; from which it would appear that the lessons taught by the

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 334.

Mr. Prichard. tests are not very definite, or that they have not been generally understood.

A knowledge of the principles involved is of great importance, both as a guide to the making of useful tests and as a key to understanding the phenomena observed. The theory of columns has been partially developed by correct analysis, but it has frequently been elaborated so much that the essential facts have been buried under what Trautwine called "heaps of mathematical rubbish." It may be well, therefore, to present a concise analysis of the influence of length and eccentricity on the strength and stiffness of columns.

Consider a column with frictionless hinged ends, of length, l, and radius of gyration, r, with constant cross-sectional area, A, subjected to a longitudinal load, of intensity, p, acting with an intentional eccentricity, e, and an accidental eccentricity, e'.

In consequence of the eccentricity, there will be a primary intentional bending moment, $p\ A\ e$, a primary accidental bending moment, $p\ A\ e'$, and a secondary bending moment, $p\ A\ \Delta$; Δ being the deflection. The value of Δ can be obtained from the well-known equation:

$$\Delta = \frac{\text{moment } l^2}{C\ E\ A\ r^2} \qquad \text{(1)}$$

in which E is the modulus of elasticity and C a factor which varies with the shape of the moment diagram.

The moment diagram can be divided into two parts, the diagram of primary moments and the diagram of secondary moments. For the determination of the deflection due to the secondary moments, the value of C will vary between limits of which the upper is π^2 and the lower depends on the form of the primary moment diagram: if it is a rectangle, the lower limit will be 9.6, while, if it is in the shape of a bow, the lower limit will approach very close to the upper limit, $\pi^2 =$ about 9.87. The limits are so narrow that it can be taken as π^2 without serious error.

For the determination of the deflection directly due to the primary moment, the value of C will vary according to the conditions, but, for convenience, it may be designated $\frac{\pi^2}{z}$. (If the cause of the primary bending moment is the eccentric application of the load, z will equal 1.234, and C will equal 8; but, if the cause is a bow-shaped bend in the axis of the column, z will be approximately equal to unity. In the applications made subsequently in this discussion, z is taken as equal to 1.234, which, in some cases, is a trifle high. The resulting stresses and deflections, therefore, are a trifle high, especially for the higher ratios of l to r.)

Substituting the primary intentional, primary accidental, and secondary moments in Equation 1 gives

$$\Delta = \frac{z\ p\ A\ l^2\ (e + e')}{A\ \pi^2\ E\ r^2} + \frac{p\ A\ \Delta\ l^2}{A\ \pi^2\ E\ r^2} \qquad \text{(2)}$$

Mr. Prichard.

To simplify the development, let

$$q = \frac{\pi^2 E r^2}{l^2} \qquad \text{.......................}(3)$$

This is Euler's formula, and, as it facilitates the application of the final equations to have the values of q, which may be termed a modulus of rupture, determined for various values of $\frac{l}{r}$ and tabulated, Table 3 is submitted.

TABLE 3.—VALUES FOR MODULUS OF BUCKLING.*

Values of the modulus of buckling, $q = \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2}$

$E = 29\,000\,000$ lb. per sq. in.

l = length, in inches. r = radius of gyration, in inches.

$\frac{l}{r}$	Values of q.	$\frac{l}{r}$	Values of q.	$\frac{l}{r}$	Values of q.
2	71 555 000	82	42 567	162	10 906
4	17 889 000	84	40 564	164	10 642
6	7 950 500	86	38 700	166	10 387
8	4 472 200	88	36 960	168	10 141
10	2 862 200	90	35 336	170	9 904
12	1 987 600	92	33 816	172	9 675
14	1 460 300	94	32 393	174	9 454
16	1 118 000	96	31 057	176	9 240
18	883 390	98	29 802	178	9 034
20	715 550	100	28 622	180	8 834
22	591 360	102	27 511	182	8 641
24	496 910	104	26 463	184	8 454
26	423 400	106	25 473	186	8 273
28	365 080	108	24 549	188	8 098
30	318 020	110	23 655	190	7 929
32	279 510	112	22 817	192	7 764
34	247 590	114	22 024	194	7 605
36	220 850	116	21 271	196	7 451
38	198 210	118	20 556	198	7 301
40	178 890	120	19 876	200	7 155
42	162 260	122	19 230	202	7 015
44	147 840	124	18 615	204	6 878
46	135 260	126	18 029	206	6 745
48	124 230	128	17 469	208	6 616
50	114 490	130	16 936	210	6 490
52	105 850	132	16 427	212	6 368
54	98 155	134	15 940	214	6 250
56	91 269	136	15 475	216	6 135
58	85 083	138	15 029	218	6 023
60	79 506	140	14 608	220	5 914
62	74 459	142	14 195	222	5 808
64	69 878	144	13 803	224	5 704
66	65 707	146	13 427	226	5 604
68	61 899	148	13 067	228	5 506
70	58 412	150	12 721	230	5 411
72	55 212	152	12 388	232	5 318
74	52 268	154	12 068	234	5 227
76	49 553	156	11 761	236	5 139
78	47 045	158	11 465	238	5 053
80	42 722	160	11 180	240	4 969

* From *Proceedings*, Engineers' Society of Western Pennsylvania, July, 1907, p. 341.

Mr. Prichard. Substituting q for its value in Equation 2, and reducing, gives

$$\varDelta = \frac{p\,z\,(e + e')}{q - p} \qquad (4)$$

Hence the secondary moment is

$$p\,A\,\varDelta = p\,A\,(e + e')\,\frac{p}{q - p}\,z \qquad (5)$$

Let V = the distance from the neutral axis to the extreme fiber on the concave side of the column.

The stress from bending, in the extreme fiber on the concave side of the column, is $\frac{\text{moment} \times V}{A\,r^2}$. Hence, if f = the combined stress in the extreme fiber on the concave side,

$$f = p + \left(\frac{p\,e\,V}{r^2} + \frac{p\,e'\,V}{r^2}\right)\left(1 + \frac{p}{q - p}\,z\right) \qquad (6)$$

In investigating physical laws, the work of the study and the laboratory should be complementary—a proposition generally conceded, but practiced too little in investigating the mechanics of structures.

To substantiate Equation 4, which differs in form, but is actually similar to one given by Moncrieff,* a comparison is submitted between the deflections calculated therefrom and the actual deflections in tests made at the Watertown Arsenal† of four wrought-iron columns; two consisting of two 8-in. channels and one 12-in. cover-plate; and two of two 10-in. channels and one 13-in. plate. In both cases the columns were latticed on the side opposite to the plate, the length was 74 radii of gyration, the load was applied by 3½-in. pins at right angles to the webs of the channels, and placed in the center of gravity of the channels. The modulus of elasticity was assumed as 27 000 000 lb., and the value of q correspondingly determined as 48 600 lb.

Considering the fact that, in making the comparisons in Table 4, no allowance was made for unintentional eccentricity or pin friction, the agreement is as close as could reasonably be expected. Moncrieff, in connection with his equation for deflection, previously referred to, gives a large number of comparisons between deflections obtained by applying his equation, and those observed in tests, which tend strongly to establish its substantial accuracy.

The maximum compression in the extreme fiber of the column bearing the test number 1632, has been calculated for the ultimate load by Equation 6, which gives 36 600 lb. per sq. in.

The amount of the unintentional eccentricity will fluctuate greatly in practice, but, in devising rational formulas for use in designing, either by direct application or through empirical formulas founded on them, the greatest amount which is reasonably possible with ordinary care should be assumed. The amount assumed should cover inac-

* *Transactions*, Am. Soc. C. E., Vol. XLV, p. 359.

† Reports for 1883, pp. 167 and 168; and 1884, pp. 54 and 55.

TABLE 4.

Mr. Prichard.

Load, in pounds.	Test 1632: e = 1.62 in. A = 17.57 sq. in. Deflections, in inches. Calculated.	Actual.	Test 1633: e = 1.64 in. A = 17.72 sq. in. Deflections, in inches. Calculated.	Actual.
10 000	0.02	0.00	0.02	0.00
20 000	0.05	0.01	0.05	0.01
50 000	0.12	0.10	0.12	0.09
100 000	0.27	0.22	0.27	0.21
200 000	0.61	0.54	0.61	0.51
250 000	0.83	0.77	0.83	0.74
280 000	0.97	0.95	0.97	0.91
300 000	1.08	1.15	1.08	1.20
306 000	......	Ult. Load.	......	......
307 000	......	......	......	Ult. Load.
310 000	1.14	......	1.14	......

Load, in pounds.	Test 350: e = 1.7 in. A = 12.48 sq. in. Deflections, in inches. Calculated.	Actual.	Test 351: e = 1.3 in. A = ... Deflections, in inches. Calculated.	Actual.
10 000	0.04	0.0	0.03	0.0
20 000	0.07	0.03	0.06	0.2
50 000	0.19	0.14	0.17	0.12
100 000	0.41	0.37	0.38	0.32
150 000	0.69	0.67	0.64	0.58
180 000	0.89	0.90	0.83	0.82
200 000	1.03	1.20	0.98	1.05
202 700	......	Ult. Load.	......	......
208 200	......	......	......	Ult. Load.
210 000	1.11	......	1.06	......

curacies in boring pin holes, the shift in the position of the axis from over-runs, and shortages in sectional area as compared with the area of the sizes specified, inequalities in the modulus of elasticity in different parts of the cross-section, curves or kinks in the axis, and potential curves or kinks in the axis from the relief of internal stresses. Owing to internal stresses produced by cold-straightening or otherwise, the metal is likely to be overstrained in spots before that in the main body of the column reaches the elastic limit. The internal stresses may be relieved to some extent by overstraining followed by a rest, but the column is likely to have a permanent deflection as a result thereof. From some causes, such as inaccuracies in pin holes, short columns are likely to have as much accidental eccentricity as long ones; while, from other causes, such as initial curvature of the

Mr. Prichard. axis, the probable limit of eccentricity will vary with the length. The following value for the factor, $\frac{e' V}{r^2}$, in Equation 6 is suggested:

$$\frac{e' V}{r^2} = \frac{1}{10} + \frac{l}{700\,r} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

For ordinary built columns with pin connections, in which the relation of V to r is about as 7 is to 5, the eccentricity corresponding to Equation 7 is:

$$e' = 0.07\,r + 0.001\,l \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

For a column with a radius of gyration of 5 and a length of 500 in., the eccentricity given by Equation 8 is about $\frac{7}{8}$ in.

From Equations 6 and 7, a formula can be deduced which will give the load, p, per unit of column area, but such a formula is not submitted, for the reasons that it is so complicated and difficult of application that it is of no practical value, and the results which it gives for columns having a length of less than 100 radii of gyration can be approximated closely by short empirical methods. The method suggested is as follows:

For structural-steel columns with hinged ends, the stress per square inch, in pounds, in the most compressed fiber, from combined direct compression and intentional primary bending moment, shall not exceed

$$\frac{35\,000 - 1.5\,\frac{l^2}{r^2}}{\text{Factor required.}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

For a factor of 2.5, the expression becomes

$$14\,000 - 0.6\,\frac{l^2}{r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

From Equations 6 and 7 it is evident that the minimum value which can be assigned for the stress from the accidental bending moment is $0.1p$, from which it follows that a limitation of 35 000 lb. per sq. in. for combined direct compression and intentional primary bending moment corresponds to a limitation of 38 500 lb. per sq. in. for stresses from all sources. Hence, if proportioning by Equation 9, with a factor of one for maximum possible loads, gives results closely in accord with theory, as claimed, the stress per square inch in columns thus proportioned, as determined by Equations 6 and 7, should be close to 38 500 lb. How close they come to this amount is shown by Table 5.

The agreement between theory and rule, indicated by Table 5, is close, except for long columns and great eccentricities, for which the rule requires a heavier column than theory.

Some opportunity for comparison between the theory and assumptions outlined, on the one hand, and experiments, on the other, is afforded by the tests of mild steel columns with pivoted ends made by Professor

TABLE 5.—MAXIMUM STRESSES PER SQUARE INCH, DETERMINED BY THEORY (EQUATIONS 6 AND 7) IN COLUMNS PROPORTIONED BY RULE (EQUATION 9, WITH FACTOR OF ONE), FOR VARIOUS LENGTHS AND INTENTIONAL ECCENTRICITIES. Mr. Prichard.

$\frac{l}{r}$	THE INTENTIONAL PRIMARY BENDING STRESSES, IN TERMS OF THE LOAD, ARE GIVEN AT THE HEAD OF EACH COLUMN.			
	0	0.1*p*.	0.5*p*.	*p*.
	Pounds.	Pounds.	Pounds.	Pounds.
0	38 500	38 200	37 300	36 750
25	39 100	39 000	38 100	37 300
50	39 100	39 200	38 600	37 500
75	39 400	39 800	38 500	36 300
100	38 700	38 800	33 900	30 700

Tetmajer.* Of the columns tested, the lengths of 27 did not exceed 100 radii, and they were loaded without intentional eccentricity. The ultimate load in all cases was greater than indicated by Equation 9, and less than required under ideal conditions for an elastic limit of 41 000 lb. and a modulus of elasticity of 29 000 000 lb. Table 6 is a comparison of the ultimate loads given by Equation 9 and the lowest of the ultimate loads shown by the tests.

TABLE 6.

$\frac{l}{r}$	Equation 9. Pounds.	Tests. Pounds.	$\frac{l}{r}$	Equation 9. Pounds.	Tests. Pounds.
30	33 650	39 000 about.	70	27 600	29 000 about.
40	32 600	38 000 "	75	26 560	28 000 "
50	31 250	32 000 "	92	22 300	23 000 "

For columns of greater length than 100 radii of gyration, stiffness rather than strength is the governing consideration. For this reason, the loads allowed by Equation 9 are preferable to those allowed by the theory of column strength.

To show the relative stiffness of columns 100 radii and longer in length, when proportioned by Equation 9 with a factor of one, the deflections have been determined by Equations 4 and 8 for columns without intentional eccentricity, with a radius of gyration of one, and various lengths as shown in Table 7.

It will be noticed that the loads allowed by Equation 10 for columns up to a length of 100 radii of gyration are about one-sixth greater than those allowed by the author, but that there is a radical difference for

ransactions, Am. Soc. C. E., Vol. XLV, p. 404.

Mr. Prichard. longer columns. The objection among engineers to columns longer than 100 radii is largely sentimental. For the same load, a column with a length of 120, 130, or 140 radii, proportioned by Equation 10, in consequence of its greater sectional area, is stiffer and stronger than one of a length of 100 radii.

TABLE 7.

Length, in inches.	Deflection, in inches.	Ratio of deflection to length.	Length, in inches.	Deflection, in inches.	Ratio of deflection to length.
100	0.486	1:206	120	0.485	1:247
110	0.550	1:200	130	0.327	1:400

For derricks, poles, and other equipment used in building and erecting, much longer struts are used than in bridge work, and, when the loads are kept within rational limits, the flexibility of the struts, by permitting them to absorb impact, is an element of safety. In the ordinary affairs of life, long struts are used as a matter of course. The engineer who becomes alarmed at long struts in structures will bear his whole weight on a walking-stick, many times as flexible as steel, with a ratio of l to r of 200. Ample provision should be made in horizontal and inclined struts for the stresses from their own weight. The frequent neglect to make such provision in long struts has doubtless had something to do with the prejudice against them.

As regards columns, the greatest need for caution to-day is in proportioning short stiff ones, which, to an engineering public accustomed to gauge permissible unit stress by the ratio of length to radius of gyration, have an appearance of strength not borne out by their details, and, if their ends are square or fixed, they are subject to strains from imperfect butt joints, or to secondary stresses produced by the deformation of connecting floor-beams, etc. Such stresses are greater for short than for long columns, on account of their greater stiffness. In consequence of these facts, it is suggested that the average load from direct compression per square inch of cross-sectional area should not exceed 13 000 lb.

With the double requirement of Equation 10 and a 13 000-lb. limitation for direct compression, if the permissible loads are plotted to a scale for various ratios of l to r, the line indicating the maximum permissible loads will suddenly change its direction at a length somewhat less than $41r$, depending on the amount of the primary bending moment. This is a feature which the author seems to consider objectionable. It is entirely natural, however. Radically different conditions govern the strength of very short and very long columns, and the loci representing the loads under these radically different condi-

tions will intersect sharply. If there is no intentional primary bending moment from eccentricity or transverse loading, 13 000 lb. per sq. in., unreduced, will govern for columns of shorter length than $41r$, and Equation 10 for columns of greater length, but if there is an intentional bending moment, both requirements should be applied to determine the governing one. Mr. Prichard.

One of the assumptions from which Equation 10 was developed was that of frictionless hinged ends. When there is no primary bending moment, any friction on the pins, according to strict theory, will fix the ends; hence, it is not surprising that friction in pins is very potent in increasing the resistance of columns to direct load in carefully devised and conducted tests. Such friction, however, is a very poor reliance in practice, as it may be overcome by a little eccentricity or shock.

Under ideal conditions, a column with strictly fixed ends has the same strength as a column of half its length with the same cross-section and pivoted ends. In practice, however, the assumption of fixed ends is never wholly realized. The appearance of having the ends fixed is frequently deceptive, as compression members on opposite sides of a joint may deflect in opposite directions in such a way that the point of contrary flexure comes very near to the center of the joint, which condition is equivalent to pivoted ends. For this reason, no easement in the reduction for length, as given in Equation 10, is recommended in designing new columns with seemingly fixed ends. In determining the safe strength of a column in an existing structure, however, if it is evident that the ends are well fixed, it might be assumed that the column is as stiff as it would be if it were about three-fourths as long and had frictionless hinged ends. For such an assumption, Equation 9 would become:

Allowed compression per square inch in any fiber from combined direct compression and intentional primary bending moment for columns with ends fixed equals

$$\frac{35\,000 - 0.85\,\dfrac{l^2}{r^2}}{\text{Factor required.}}$$

In comparing this discussion with the writer's paper,* entitled "The Proportioning of Steel Railway Bridge Members," it will be noticed that the greatest compression now recommended is one-fifteenth less than in the paper referred to, in addition to which a limitation for direct compression to 13 000 lb. per sq. in. is now recommended. This change is the result of a further consideration of the subject, in the light of the Quebec Bridge disaster and the general discussion regarding columns which followed it, including the paper by Mr. Worcester.

**Proceedings*, Engineers Society of Western Pennsylvania, July, 1907.

Mr. Prichard. It should be stated, however, that it has always been the writer's practice, in designing columns, to give close attention to the details and the make-up of the section, and to make a liberal reduction in the allowed unit stresses when the unsupported width of a plate exceeded 32 times its thickness. The following requirement as to the unsupported width of plates is suggested:

If the unsupported width, w, of any plate in a column is more than 32 times its thickness, t, the permissible stress, as given by Equation 10, shall be reduced by multiplying it by the following expression:

$$\text{Permissible stress, as given by Equation 10,} \times \frac{16\,000 - \dfrac{2\,w^2}{t^2}}{14\,000}.$$

Mr. Horton. HORACE E. HORTON, M. AM. SOC. C. E. (by letter).—Mr. Worcester's compilation of results of tests on full-sized wrought-metal compression members is very interesting and instructive, and is opportune. There is an awakened interest in the subject at this time.

While the writer approves unhesitatingly Mr. Worcester's criticism on using excessive unit stress on members with short radii lengths, he knows no physical reason for limiting compression members to a length of 100 radii.

Mr. Worcester has chosen to make his platting of tests for steel on the basis of four-fifteenths of the ultimate strength. For obvious reasons, the writer uses the same. Mr. Worcester has used 12 000 lb. per sq. in. as his unit value in compression, and the writer naturally uses the same stress, with the reservation that the unit stress (tension) is $1\frac{1}{3} \times$ the compression, that is, 16 000, in this case.

The diagram, Fig. 2, gives all the tests of steel members shown by Mr. Worcester, also tests of six members* by J. A. L. Waddell, M. Am. Soc. C. E., and seven tests† by Mr. C. P. Buchanan, and, further, six tests by the Chicago Bridge and Iron Works, on 8 by 8 by $\frac{7}{16}$-in. angles.

On this diagram a straight line is drawn through the center of gravity of the group of tests, and is expressed by $11\,300 - 35\frac{l}{r}$, also the formula for loading, as indicated by C. L. Strobel, M. Am. Soc. C. E., in his paper, "Experiments Upon Z-Iron Columns,"‡ wherein was first laid down the necessity of "sawing off" the unit stress for short radii length, in this case to 8 000 lb. per sq. in., and also the first appearance of the straight-line formula, which was $10\,600 - 30\frac{l}{r}$.

The writer has platted Mr. Worcester's formula, based on 12 000, and, as a protest against the attempted "amputation" for radii lengths

* The record of these tests appeared in *Engineering News*, January 16th, 1908.

† *Engineering News*, December 26th, 1907.

‡ *Transactions*, Am. Soc. C. E., Vol. XVIII, p. 103.

Mr. Horton.

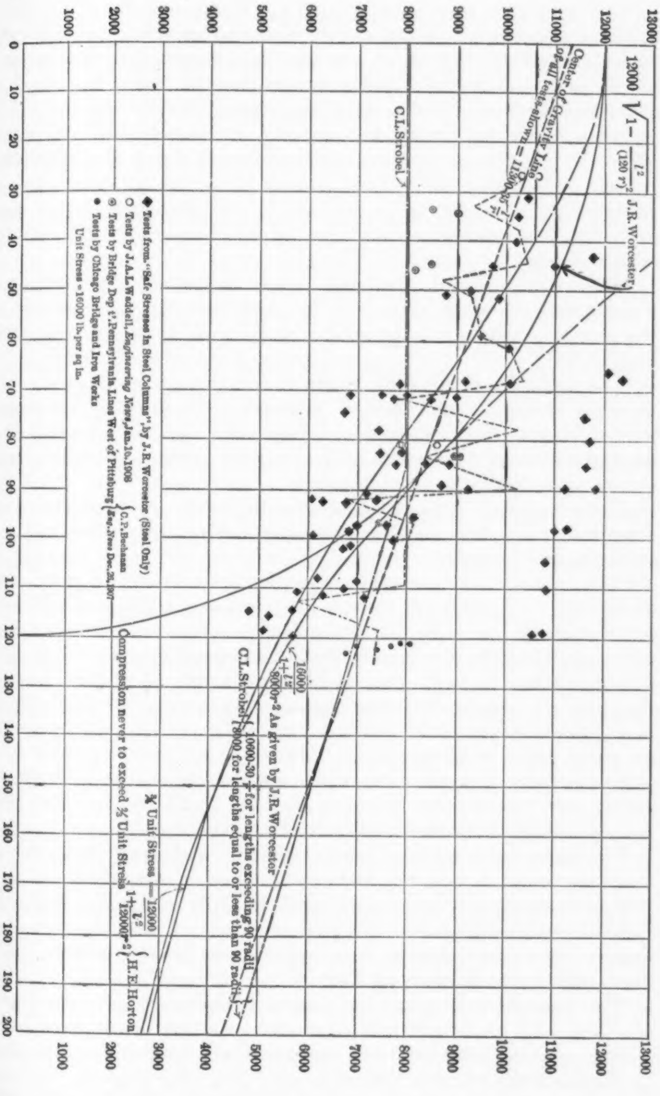


Fig. 2.

Mr. Horton. of more than 100, there is also platted the Hodgkinson-Gordon-Rankine formula, as given by Mr. Worcester, based on 16 000, with a lower divisor of 8 000. It will be noticed that the platted lines come tangent at 85 radii, and the two curves come somewhat above the center of gravity of the tests. However, as these values are high for short radii lengths, clearly indicating the necessity of "sawing off," the writer offers the Hodgkinson-Gordon-Rankine formula, based on 12 000, with a lower divisor of 12 000. When "sawed off" at two-thirds of the unit stress (that is, 10 666 lb. per sq. in. as the ultimate value in compression), and intersecting the curve at 40 radii length, it looks both sane and safe.

The curve produced as platted, clearly indicates what is intended, a value well below the average of the tests, and the value reduced so that one may believe it to be reliable as the radii lengths are extended, even to 200.

The limitation to 100 radii length of compression members can only be urged because of our want of knowledge, but practice and experience show that the greatest hazards are with short radii lengths, due to want of proper proportion and cohesion of parts, and the tendency toward using material which is too thin.

Unfortunately, engineering discussion as to the unit value of compression members has been almost entirely on formulas, and not on the physical column.

As to the physical compressive member, Mr. Buchanan gives a report, with full details of tests to destruction, of nineteen full-sized bridge members as built for actual use in structures—twelve of iron and seven of steel. The first noticeable thing is that radii length has no significance, in fact, members having a length of 83 radii were as strong as any tested, and much stronger than many of less than 40 radii, and members of 97 and even 120 radii were a good average in the whole group of tests.

The average ultimate strength of seven steel columns is 31 900 lb. per sq. in. The average crippling strength is 23 800 lb. per sq. in.; the average elastic limit is 19 700 lb. per sq. in.

The ultimate strength, crippling strength, and elastic limit, in the foregoing tests as reported, indicate a value of scarcely more than 50% of the value of the steel in tension. This is startling, with our knowledge of specifications permitting the use of steel in short radii lengths for approximately the same stress as in tension. Mr. Buchanan's tests are given in Table 8.

The four **Z**-bar columns lack, on an average from the computed crippling load, essentially half as much as did the fifteen trough and channel sections (the last with less than half the radii length), and yet they actually stood a greater load.

From the photographs of the members taken after the tests, it is

seen that four Z-bar column struts yielded as a whole by flexure. The fifteen other members yielded by some order of wrinkling or failure in individual parts. It is further noticeable that the sections of the Z-bar columns were much thicker, relatively. It is clearly apparent that compressive members which fail by wrinkling fail at less load per unit than those which fail by flexure. Mr. Horton.

TABLE 8.

Average of Buchanan's tests.	$\frac{l}{r}$	Average of T. H. and J. B. Johnson's formula. Crippling load.	Actual crippling load.	Below estimate.
4 tests, Z-bar columns..........	96	28 537	21 700	6 837
15 tests, trough and channel columns	43.6	33 125	20 730	12 395

In the photographs of the material, after the tests by Mr. Waddell referred to above (all of the same cross-section), it is noticeable that members 81 radii long failed by flexure while those 27 radii long failed by wrinkling. From the Buchanan tests there is abundant evidence to conclude that the best results are obtained when the member yields by flexure. From Mr. Waddell's tests there is evidence that, for the best results at 27 radii, the material must needs be thicker than for 81 radii. Here is a suggestion, to be enlarged on later.

While the radius of gyration has use, as indicating the value of a strut, there is much to show that there are many other conditions of as great importance as the radii length.

The radius of gyration will modify and hold in check any disposition to use material of undue thickness, but the radius of gyration has to be held in check unless too thin material be used. The radius of gyration of a transverse element of the column may be used as such a check.

The composite nature of the compression member directly reduces its unit value, as compared with tension, a very material amount. This is directly traceable to the possible rivet efficiency connecting parts, and it is undoubtedly a fact that rivets are driven much too far apart. Two or three times as many rivets would surely give better results. Rivet connections between multiple plates, or plates and angles, forming a compression member, to reduce the tendency to wrinkling, are clearly different from tension connections, and the efficiency has to be considered locally.

With material half as thick as the rivet diameter, an efficiency of 50% may be obtained by pitching the rivets at 2.3 diameters; but, with material the thickness of the rivet diameter, and rivets pitched at 2 diameters, an efficiency of connection of 30% is all that is possi-

Mr. Horton. ble. In practice, rivets are generally driven with three times as much pitch as here indicated, and the assertion may be made that the efficiency of the connection of parts by rivets scarcely exceeds 12 per cent.

The cross-section of the compression member is unquestionably of great significance. The proportions of the material in the flanges and its width and thickness undoubtedly have paramount importance.

The compression member, with ever-increasing tendency in the evolution of design, has developed with one or more open sides on which lattice bars are used.

The proportions of such lattice bars, their connections to the columns, and their relation to a force acting through the compression member form a very material and important element, second to none in the design. At the present time, there are in the technical press many letters from correspondents, with elaborate formulas in which E represents the modulus of elasticity, and e the eccentricity. As e, eccentricity, is arbitrarily assumed, the writer prefers to assume a percentage of the compression through the column, and call it shear.

The difference between the eccentricity discussed and the shear outlined is as follows: Eccentricity is an assumption without reference to the magnitude or amount of the force acting on the member, while the shear is a direct percentage of the force acting on the member. One leads to the discussion of how accurate the workmanship of the column may be, or is. The other asserts the fact that there must be some relation between the force acting through a compression member and its disposition to "side-step." This uncertainty is not caused by faulty workmanship, but comes from a want of research and knowledge.

In all the years past the whole discussion and the specifications for compression members have absolutely ignored both shear and eccentricity as items to consider, except in what has appeared within a very short period, and there is no evidence that our workmanship has especially deteriorated in the immediate past, but there is reason to hope that our knowledge of design may be enlarged.

Figs. 3 to 15 are given in order to indicate to the eye the relation of various sections expressed by the radius of gyration; each section has the same cross-section, namely, 12 sq. in.

Fig. 3 is a solid, 3.46 in. on a side, radius of gyration = 1.

Fig. 4 is a hollow square, 5 in. on a side, metal $\frac{5}{8}$ in. thick, radius of gyration = 2.

Fig. 5 is a hollow square, $7\frac{3}{4}$ in. on a side, metal $\frac{13}{32}$ in. thick, radius of gyration = 3.

Fig. 6 is a hollow square, $10\frac{1}{2}$ in. on a side, metal $\frac{19}{64}$ in. thick, radius of gyration = 4.

Fig. 7 is a hollow square, $12\frac{3}{4}$ in. on a side, metal $\frac{15}{64}$ in. thick, radius of gyration = 5.

Mr. Horton.

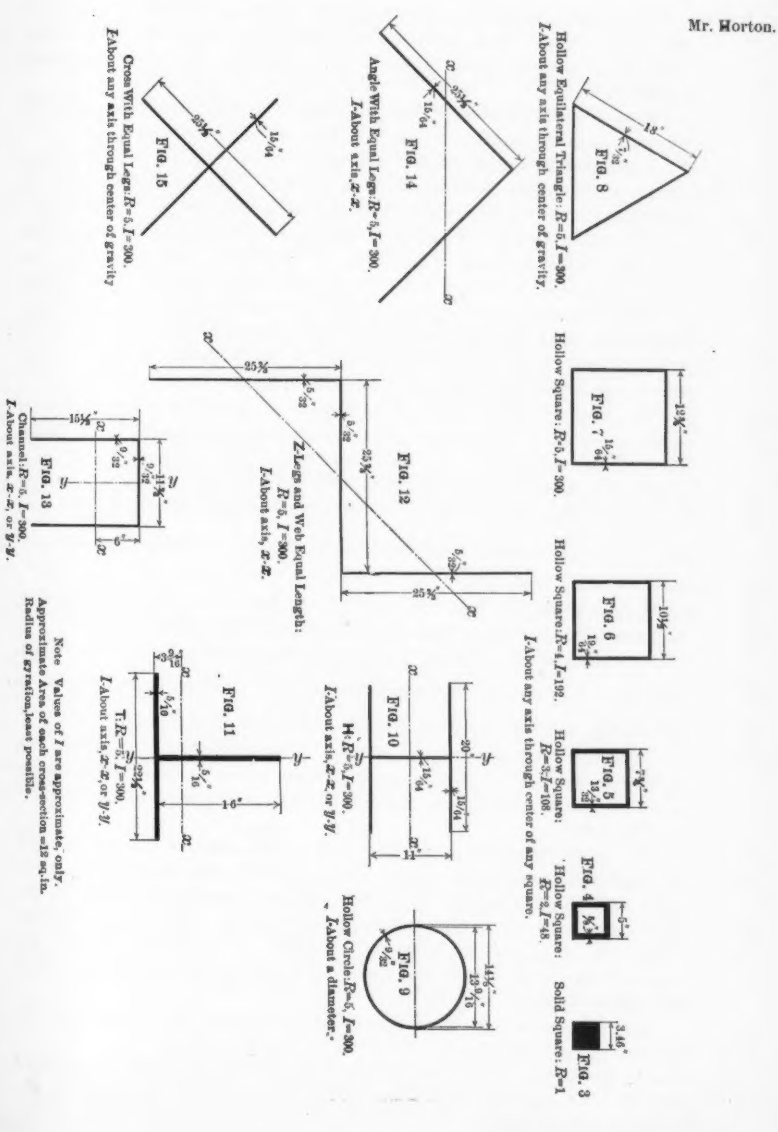


FIG. 3
Solid Square: R=1
3.46"
FIG. 4
Hollow Square: R=2, I=48.
5"
FIG. 5
Hollow Square: R=3, I=108.
7⅞"
FIG. 6
Hollow Square: R=4, I=192.
10⅞"
FIG. 7
Hollow Square: R=5, I=300.
12⅞"
15/64"
I-About any axis through center of any square.
FIG. 8
Hollow Equilateral Triangle: R=5, I=300.
I-About any axis through center of gravity.
18"
7/32"
FIG. 9
Hollow Circle: R=5, I=300.
I-About a diameter.
14⅛"
13 9/16"
9/32"
FIG. 10
H: R=5, I=300.
I-About axis, x-x, or y-y.
20"
11"
15/64"
FIG. 11
T: R=5, I=300.
I-About axis, x-x, or y-y.
16"
22⅛"
5/16"
3 9/16"
FIG. 12
Z-Legs and Web Equal Length: R=5, I=300.
I-About axis, x-x.
25⅜"
5/32"
FIG. 13
Channel: R=5, I=300.
I-About axis, x-x, or y-y.
15⅛"
11⅞"
9/32"
6"
FIG. 14
Angle With Equal Legs: R=5, I=300.
I-About axis, x-x.
25⅛"
15/64"
FIG. 15
Cross With Equal Legs: R=5, I=300.
I-About any axis through center of gravity.
25⅛"
15/64"
Note Values of I are approximate, only.
Approximate Area of each cross-section = 12 sq. in.
Radius of gyration, least possible.

Mr. Horton. There are changes in the radius of gyration of from 1 to 5, with the same cross-section, with a diminishing thickness of the material, and an increasing unit value of the material by all compression formulas.

Figs. 8 to 15, inclusive, are interesting as indicating 12 sq. in. of section, in quite familiar shapes, with a radius of gyration of 5.

Appended to compression formulas it is quite usual to find a limitation of thickness to width, of 1 to 30, and Figs. 8 to 15 can only be objected to on this limitation, and not as to the radius of gyration.

According to Mr. Worcester's curve, as platted for working loads on columns 10 ft. long, Figs. 7 to 15, inclusive, may be worked for 11 800 lb. per sq. in.; Fig. 6 for 11 700 lb.; Fig. 5 for 11 300 lb.; Fig. 4 for 10 400 lb. per sq. in.; and Fig. 3 for zero. With this conclusion the writer does not agree. Figs. 4 or 5, undoubtedly, will carry the largest load of any of the sections, 3 to 15, inclusive, at 10 ft. long, while Fig. 3 will undoubtedly be a close second.

The writer would extend the radius of gyration to the elements making up the cross-section of a column, thereby limiting the thickness of the material. The radii length on a transverse section of a plate should never be more than the radii length of the column of which the plate is a part, or if it is, such parts should be used at a decreased unit stress, found by substituting the value of $\frac{l}{r}$ thus obtained in the general formula.

Angles should be one-fifth the size of the transverse dimensions of a member, and not less than the thickness of the plates.

In the case of columns with projecting portions, such as angles, Z's, etc., $\frac{l}{r}$ (where l = projection and r = radius corresponding to thickness) must be doubled and substituted in formulas.

Where a part is made of several thicknesses riveted together, the transverse radius of such a part will be taken as the radius of the same as though solid and divided by the square root of the number of pieces used.

The radii length of a lattice panel or the pitch of the lattice, with the radius of gyration of neutral axis parallel with central line of the web, of a built channel or similar section, should never exceed the radii length of the entire member. The lattice need not exceed two diameters of the rivet. The radii length of the lattice between the connections should not exceed the radii length of the member on which the lattice is used.

The lattice should have the ability to carry shear, assuming the column to be supported at its two ends or in the center:

1.—At the unit strains allowed in the column itself, an assumed uniform load equal to 10% of the load sustained by the column;

2.—At a unit stress of half the above, the weight of the column itself. Mr. Horton.

The writer wishes at this time to emphasize his faith and belief in proportion—the "Rule of Three" of our ancestors. It is the fundamental basis of comparison in all things.

Table 9 is an outline for five 2-built channel lattice columns.

Each column is in exact proportion, by the ratio of 2, in all its three (and more) dimensions, to the next of the series. It follows at once that the cross-section of the columns will be as the square, and, for the same radii length, their weight as the cube. The writer has outlined for the center of this group of five columns a rationally proportioned 12-in. 2-built channel column having a section of 23 sq. in.; he has also doubled it, and doubled it again. He has also divided the 12-in. 2-built channel column in each of its dimensions by 2 and by 4, and in this tabulation by direct proportion there are five 2-built channel columns. There is every reason to believe that the 3-in. 2-built channel column, at $\frac{1}{4\,096}$ part of the weight for a proportional radii length of the 48-in. 2-built channel column, can be investigated with reasonable certainty as to any in the group of columns that are in direct proportion in all their elements, that is, size of rivets, size of lattice, and pitch of rivets; and it is in this way that research can be carried out at comparatively trifling cost. With the testing machines already available, the truth can be developed as to any or all of the moot questions as to value of cross-sections and radii length.

TABLE 9.

Built channel columns.	Section.		Area, in square inches.	Radius.	Rivets.	Double lattice.	R. of gyration, neutral axis, parallel to center line of web.	Length. $\frac{100\,l}{r}$ for
	Plates.	Angles.						
3-in.	$2 - 3 \times \frac{1}{8}$	$4 - \frac{3}{4} \times \frac{3}{4} \times \frac{1}{8}$	1.44	1.08	$\frac{3}{16}$-in.	$\frac{3}{8} \times \frac{1}{8}$	0.22	9 ft.
6-in.	$2 - 6 \times \frac{1}{4}$	$4 - 1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	5.75	2.17	$\frac{3}{8}$ in.	$\frac{3}{4} \times \frac{1}{4}$	0.43	18 ft.
12-in.	$2 - 12 \times \frac{1}{2}$	$4 - 3 \times 3 \times \frac{1}{2}$	23	4.33	$\frac{3}{4}$-in.	$1\frac{1}{2} \times \frac{1}{2}$	0.86	36 ft.
24-in.	$2 - 24 \times 1$	$4 - 6 \times 6 \times 1$	92	8.66	$1\frac{1}{2}$-in.	3×1	1.73	72 ft.
48-in.	$2 - 48 \times 2$	$4 - 12 \times 12 \times 2$	368	17.32	3-in.	6×2	3.55	144 ft.

Table 10 is a second compilation for five columns having the same areas and dimensions as the columns in Table 9.

The columns in Table 10 will not require a testing machine, because when we have divided down from 48-in. 2-built channel columns to the 12-in. 2-built channel columns, 23 sq. in. in area, and find four 12-in. plates massed together making 12 by $\frac{7}{8}$ in. of metal combined

Mr. Horton. with $1\frac{1}{8}$ by $1\frac{1}{8}$ by $\frac{1}{4}$-in. angles with 1 by $\frac{3}{16}$-in. lattice, all secured by $\frac{1}{4}$-in. rivets, common sense will indicate that the columns in this group should not be used.

TABLE 10.

Built channel columns.	Section.		Area, in square inches.	Radius.	Rivets.	Double lattice.	R. of gyration, neutral axis, parallel to center line of web.	Length, for $\frac{100\,l}{r}$
	Plates.	Angles.						
48-in.	$8 - 48 \times \frac{7}{8}$	$4 - 4\frac{1}{2} \times 4\frac{1}{2} \times 1$	368	14.82	1-in.	$4 \times \frac{3}{4}$	1.38	124 ft.
24-in.	$8 - 24 \times \frac{7}{16}$	$4 - 2\frac{1}{4} \times 2\frac{1}{4} \times \frac{1}{2}$	92	7.41	$\frac{1}{2}$-in.	$2 \times \frac{3}{8}$	0.69	62 ft.
12-in.	$8 - 12 \times \frac{7}{32}$	$4 - 1\frac{1}{8} \times 1\frac{1}{8} \times \frac{1}{4}$	23	3.71	$\frac{1}{4}$-in.	$1 \times \frac{3}{16}$	0.35	31 ft.
6-in.	$8 - 6 \times \frac{7}{64}$	$4 - \frac{9}{16} \times \frac{9}{16} \times \frac{1}{8}$	5.75	1.85	$\frac{1}{8}$-in.	$\frac{1}{2} \times \frac{3}{32}$	0.17	16 ft.
3-in.	$8 - 3 \times \frac{7}{128}$	$4 - \frac{9}{32} \times \frac{9}{32} \times \frac{1}{16}$	1.44	0.93	$\frac{1}{16}$-in.	$\frac{1}{4} \times \frac{3}{64}$	0.09	8 ft.

The "Rule of Three" may be accepted as an agent, to assist in approaching the testing machine with columns of a size and cost so that we may hope for extended research. The "Rule of Three" may also be accepted as an agent to assist our common sense, as shown in the second compilation.

In the foregoing, the writer has attempted to point out the desirability of using all the rivet section possible in combining the parts of a composite compression member.

All research which is available indicates that the thickness of the material in the rectangular compression member has most to do with its efficiency, thick material being required for short radii length, and reducing in thickness as the radii length increases.

The piling together of relatively thin plates in multiple, with a few tack rivets, and assuming that the mass is homogeneous is dangerous.

Some comprehensive proportion of stress through the compression member must be accepted as shear, and must be provided for; if, on a 12 or 15-in. 2-channel strut of medium size, practice dictates lattice of a weight equal to, say, 30% of the scantling weight of the member, the "Rule of Three" will indicate that these same relations must be extended to large or small members.

It is not formulas that are needed to extend our knowledge of the compressive member, but comprehensive research by physical tests.

Mr. Shearwood. F. P. Shearwood, M. Am. Soc. C. E. (by letter).—Mr. Worcester's curve appears to be more rational than any of the others he has plotted; still, in common with all column formulas, his assumes that the flexural stresses only result from the tendency of the member as a whole to bend, and no reduction is allowed for the secondary bending from the unsupported component parts and other unavoidable bending

stresses which occur in many of the compression sections now in use, and especially in those having radii of gyration relatively large in comparison with their areas. Mr. Shearwood.

Strict adherence to a specified column formula has perhaps done very much to force designers to use compression members which are undesirable in nearly every way except that they meet the requirements of the formula economically as regards material.

All, or nearly all, specifications have called for the unit stresses in columns to be determined solely by the ratio of their length to their radius, the latter to be calculated from the moment of inertia of the section, without regard to whether the lattice (if used) is capable of developing it, or whether, in so doing, secondary stresses are induced.

The latticed double channel section with flanges turned out, so frequently used for compression members of truss bridges, is a good illustration of the incompleteness of the ordinary column formula. This section is generally used because, with a given width from out to out of chord gusset plates, it will give a strut having the largest radius, and therefore the highest permissible unit stress; but, if investigated, it will be found that the following stresses are almost inevitable in such a section, and of these the column formula takes no account, and they are practically unprovided for:

I.—Stress due to the flexure of the unsupported parts between lattice-bar connections, which is coincident with that due to the flexure of the column as a whole;

II.—Stress due to the eccentricity of the end connections, since the center of gravity of either channel is usually some considerable distance from the center of the gusset plates;

III.—Stress due to the eccentricity of the lattice-bar connections; for it is usually impracticable to arrange the bars so that they will intersect on the center of gravity of the channel;

IV.—Serious but less determinate stresses are probably induced at or near panel points, where, owing to the necessary connections, it may be impracticable to provide the last few feet of an important compression member with either tie-plates or lattice bars; and, even when center diaphragms are provided, the continuity of the lattice system is broken up, resulting in unknown bending moments in the member.

V.—In nearly all bridges, the loads are applied more or less on the inside of the trusses, thereby inducing longitudinal shear in the several members, which in turn must stress the eccentrically connected lattice bars, and increase the local stresses, as in III.

VI.—Lattice bars, having of necessity to be placed at an angle to the direct stress of the member, create a distortion and bending when the main member is under strain.

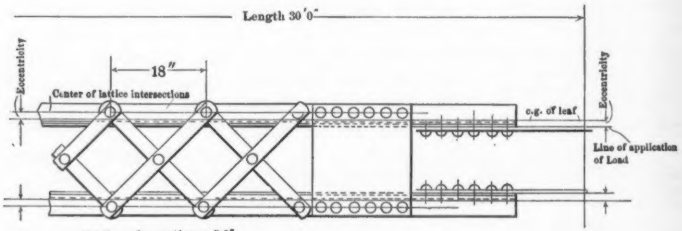
Most of the foregoing defects are absent or are minimized in members having plate diaphragms, such as **H**-shapes, which are symmetrical

Mr. Shearwood. about every axis. Secondary bending is largely eliminated, and all metal resists stress in direct lines to the applied loads. They are well adapted to transfer any uneven application of load, but, unfortunately, owing to their relatively small radius, they cannot be made to figure as economically as flimsy latticed members.

It seems to the writer that columns with their several parts tied together with solid plates should have more favorable consideration than those which are occasionally tied together with redundant and stress-inducing lattice bars.

Mr. Worcester, in common with many other authorities, apparently attributes no advantage to fixed ends over pin ends.

It would seem reasonable that members with fixed ends should have their lengths multiplied by some factor (say 0.7) when the allowed unit stress and limiting lengths are computed.



Radius of gyration = 5.9"
" " " of single leaf = 0.92"

Area of section = 33 sq. in.

Allowed stress by straight-line formula = $16000 - 70\frac{l}{r}$ = 11720 lb. per sq. in.

Permissible stress if some of the other stresses due to flexure are provided for.

Unit stress		16000
Deduct for flexure of Column as a whole	4280	
" " " " unsupported leaf	1370	
" " bending stress from eccentric lattice connection	700	6350
		9650 Permissible Stress

FIG. 16.

Fig. 16 is given as an example, and shows the direct unit stress allowed by the straight-line formula of the American Railway Engineering and Maintenance of Way Association, in which only the flexure of the column as a whole is considered, and also that permissible if the flexure of the unsupported portions of the individual leaf and bending from the eccentricity of the lattice bars are also provided for.

It seems probable that the disregarded secondary flexural stresses in latticed columns have caused the tests on short lengths to disagree with column formulas which are based on the compression value of the metal. In devising a new column formula, the many inherent weaknesses of latticed forms should be taken into account. Such a

formula would have the advantage of discouraging the use of exaggerated forms with redundant metal, and encouraging the use of members with continuously connected component parts. Mr. Shearwood.

LEWIS D. RIGHTS, ASSOC. M. AM. SOC. C. E.—This paper brings forward a subject which is of interest, not only to engineers who work with structural steel, but to all constructors who use columns of other material. Mr. Rights.

Believing that there is a feeling among a number of engineers that the "factor of ignorance" in regard to steel columns has an insufficient margin, the author considers all the available tests, reduces them to equivalent working values, and plots the results. He then breaks away from any attempt to assume a theoretical formula, and introduces a curve which agrees fairly well in its middle portion with the average of the tests, and has the limiting values of 12 000 lb. at one end and $120 \frac{l}{r}$ at the other.

As shown by the author's diagram, Fig. 1, most of the formulas now in common use indicate considerably higher values than those given by the proposed curve, and the question at once arises whether engineers are warranted in making such a radical reduction. In the light of present knowledge, the speaker does not feel that it is advisable to take such a step. Many of the tests which the author has plotted are from twenty to twenty-five years old; some of them are on plain shapes, and many of them, as indicated, are for iron. The present practice has been built up from these same tests, and is an attempt, perhaps in a makeshift way, to accommodate itself to the improved conditions of manufacture and details, which have changed materially since the tests were made. It is the speaker's belief that engineers would hardly feel justified in recommending this increased expense to their employers or clients.

Although the speaker cannot agree with the author as to the large reduction proposed, nevertheless, he feels that some enthusiasts, overconfident in the supposed knowledge concerning the present state of manufacturing, have increased the working values beyond safe limits, and he would suggest that there is a middle ground. He would like to offer as a temporary formula, and more or less of a compromise, the straight line produced by $15\,000 - 75 \frac{l}{r}$.

For the initial point, 15 000 lb. seems to be a satisfactory value. A very large proportion of the steel now used has an average ultimate stress of 60 000 lb., with an elastic limit of 33 000 lb., and the values suggested, if properly reduced, would come within what is, at present, considered a safe limit. It will be noted from the diagram, Fig. 17, that this straight line agrees with the author's plotted tests in the middle portion fully as well as the proposed curve, and that it would be practically tangent to the proposed curve at this position.

Mr. Rights. While it is desirable to cut off the column formula at some higher value of $\frac{l}{r}$, the question also arises as to where this point shall be. Some engineers favor 100, others, 120, others, 125, and all of them probably have had occasion at some time to consider columns at even a higher value of $\frac{l}{r}$ than 125. When such high values become absolutely necessary, the engineer uses the old formula with discretion. It is the speaker's belief that, instead of attempting to saw off the curve at some arbitrary point, it should be made fool-proof by giving it safe values above 120 $\frac{l}{r}$. It will be seen that the proposed straight line would end at 200 $\frac{l}{r}$, and would give fairly low values between this and 120 $\frac{l}{r}$.

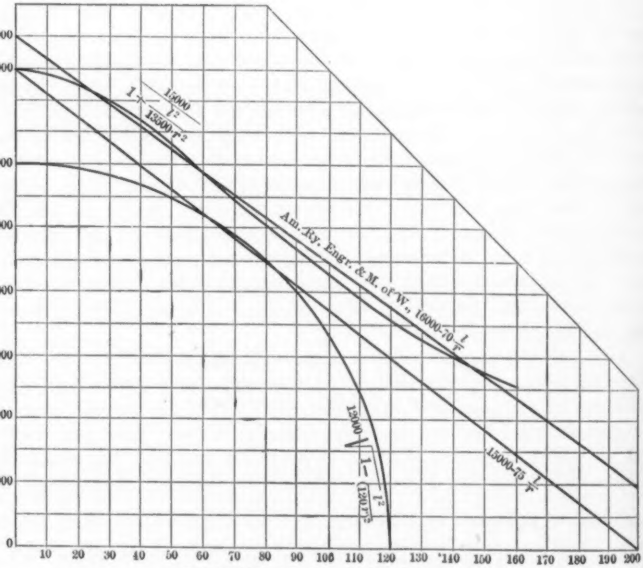


DIAGRAM OF COLUMN FORMULAS

FIG. 17.

In the light of our present knowledge, or ignorance, the straight-line formula would seem to be adequately accurate for all practical use, and the speaker feels that engineers could not do better than adopt such a simple formula until more is learned about the subject.

The speaker's suggestion would be that engineers either stick to their present formulas, using them in a conservative manner, or adopt some simple straight line, as suggested herein. New tests are needed, not new formulas, and, until these tests become available, it would be better for engineers to work conservatively along the lines they have been taught. Mr. Rights.

A. W. Carpenter, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Worcester apparently overlooked the fact that full-sized tension members do not develop the strength of specimen test pieces, and his comparison of the ultimate strength of full-sized columns with the ultimate tensile strength of test specimens, therefore, is hardly on a fair basis. In proof of the statement that full-sized tension members do not develop the strength shown by test specimens of the same material, the writer would cite the tests* by J. E. Greiner, M. Am. Soc. C. E., on built-up tension members; the "Tension Tests of Steel Angles with Various Types of End-Connection,"† by Frank P. McKibben, M. Am. Soc. C. E.; and any bridge engineer's records of tests of full-sized eye-bars. Mr. Carpenter.

TABLE 11.

Description of tests.	Specimen tests.			Full-sized tests.		Ratio of full-sized to specimen tests.	
	Material.	Yield point, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Yield point, in pounds per square inch.	Ultimate strength, in pounds per square inch.	Yield point.	Ultimate strength.
J. E. Greiner's tests of built-up tension members: Nos. 3 to 10 of Series A; Nos. 11 to 18 of Series B: Average of 16 tests of full-sized members..................	Mild steel.	38 250	58 200	45 030	50 650	1.19	0.87
J. E. Greiner's tests of single-angle tension members connected by both legs: Average of 4 tests.	Mild steel.	37 580	57 280	27 450	51 050	0.73	0.89
McKibben's tension tests of single angles connected by both legs: Average of 12 tests..............	Mild steel.	33 040	56 480	Not given.	46 750	......	0.83
Average of 70 tests of full-sized eye-bars reported in Mr. Greiner's paper	Medium steel. (?)	37 342	63 582	31 270	57 745	0.84	0.91
Average of 24 tests of eye-bars 10 by $1\frac{1}{16}$ to $1\frac{5}{16}$ in., made under the writer's direction................	"	40 187	61 630	30 440	59 160	0.76	0.96

Note.—All test specimens unannealed, except those of the last item.

Table 11 is a summation of the results of these tests in form to bring out the point desired.

* *Transactions*, Am. Soc. C. E., Vol. XXXVIII, p. 41.

† *Engineering News*, July 5th, 1906.

Mr. Carpenter. It should be stated that, of Mr. Greiner's tests on built-up members, Nos. 1 and 2 of Series A and all of Series C were excluded from Table 11 for the reason that the members all had defective (intentionally so designed) end connections; likewise, only the angles which were connected by both legs in the angle tests mentioned were considered; therefore, the results are the most favorable possible toward the best development of strength.

The high ratios of ultimate strength of eye-bars to that in specimen tests must be considered badly offset by the low yield-point ratios. The yield-point ratios for Mr. Greiner's angles are certainly not favorable to the tension side of the argument. The other tests by Mr. Greiner were made on members built of small sections, which probably accounts for the high yield-point values. Following the usual laws, lower results for yield point and (in lesser degree) for ultimate strength would be found in the members of truss bridges and other structures, which are built of thicker sections.

It would seem that, excluding eye-bars, it would be unsafe to call the average ultimate unit tensile strength of full-sized tension members more than 0.85 times the corresponding strength of test specimens of the same material; or, assuming 60 000 lb. as the ultimate unit tensile strength of structural-steel test specimens, the corresponding average strength of full-sized tension members would be 51 000 lb. Now, put Mr. Worcester's Diagram of Column Formulas on the basis of $\frac{16\,000}{51\,000}$ for steel and $\frac{16\,000}{\frac{5}{6} \times 51\,000}$ or $\frac{16\,000}{42\,500}$ for wrought iron, and the value he has chosen as the unit stress for the ratio, $\frac{l}{r} = 0$ (12 000 lb.), will become 14 000 lb. (practically).

Mr. Worcester entirely disregards the effect of end conditions on the columns tested, treating columns with flat and hinged ends alike. This seems to be rather unsatisfactory, since the theoretical influence of the end conditions on the strength of columns is well backed up by tests, and, undoubtedly, conditions arise which justify a distinction in this regard. The writer is wholly in favor of a single formula for bridge work, and that based on hinged ends, since this condition is closely approximated in pin-connected members, and the strengthening effect of greater fixity of ends in members with riveted connections is offset in unknown degree by secondary stresses and unavoidable eccentricity of loading. It would seem preferable to base a curve on, or compare formulas with, full-sized tests of columns the end conditions of which are alike. For building work and special cases, in which the condition can be unquestionably realized, a formula for columns with fixed ends would seem to be entirely proper.

Mr. Worcester mentions the tests of Tetmajer, Marshall, and

Christie as "full-sized." In his excellent paper, entitled "The Practical Column under Central or Eccentric Loads,"* Mr. J. M. Moncrieff gives separate diagrams covering all the important series of tests of columns made, up to the date of the paper, and apparently includes all the tests cited by the author. According to these diagrams, the tests of Tetmajer, Marshall, and Christie were on very small "full-sized" members, generally, such as bars 1 in. square, small angles, and other shapes and tubes. It seems that some distinction should be made between these (especially the solid bars of insignificant size) and large columns, such as those tested by Bouscaren, Strobel, and the Watertown Arsenal. It was noted that the particularly low result of 28 000 lb. ultimate strength for a column having $\frac{l}{r} = 30$ was obtained in the series of "55 tests at Watertown Arsenal of 3-in. square bars (cold-straightened), mostly on pins $1\frac{1}{2}$ in. in diameter, eight being on pins from $\frac{7}{8}$ in. to $2\frac{1}{4}$ in. in diameter," all of wrought iron. It would seem that such tests should be given very little weight in this consideration. The writer has failed to note any test of a properly-constructed, centrally-loaded large column which gives any such low result. The author's statement regarding a factor of only 2 between the ultimate strength of columns and a working stress of 16 000 lb. per sq. in. in compression seems to be misleading, because, if the 16 000 lb. be considered the constant for reduction in one of the usual column formulas, such tests as have been made on large columns show an average factor of considerably more than 2—perhaps 2.5 for mild steel—and it has been pointed out that the average factor in tension is about 3.2. The range of variation from the average is thought to be about the same in tension as in compression. A careful study of tests of large steel columns leads the writer to think that it would not be far wrong to take, as the value representing the ultimate strength of well-proportioned and properly-detailed columns, in the numerator of the Gordon-Rankine or other equivalent column formula, 41 000 lb. for mild steel and 36 000 lb. for wrought iron. These values would require, for equal factors of safety based on ultimate strength, approximately the following comparative values: Mr. Carpenter.

For tension in steel, 16 000 lb. per sq. in.
For compression in steel, 13 000 lb. per sq. in.
For tension in wrought iron, 13 000 lb. per sq. in.
For compression in wrought iron, 11 000 lb. per sq. in.

In spite of the author's remarks, it seems difficult to get around the fact that engineers do and must design with the elastic limit in view, and not the ultimate strength, and that the structure is unsafe and possibly ruined when the elastic limit (or more properly perhaps,

*Transactions, Am. Soc. C. E., Vol. XLV, p. 334.

Mr. Carpenter. the yield point) is passed, in tension as well as in compression. Also, there is considerable strength beyond the yield point in compression, which, as far as it goes, is just as valuable as the tensile strength beyond that limit. There appears to be a lack of data on the elastic strength of columns. Such as the writer has been able to find, indicate that the elastic strength will be found below that of test specimens, but not more so than with eye-bars in tension. There also appears to be much greater danger from imperfect workmanship and injuries to material, in the case of columns, than in tension members, for which reason the writer agrees with the author, that a lower unit should be used in compression than in tension, and thinks that perhaps the ratio derived from the ultimate strength values proposed, will be satisfactory, that is, a compression unit of about eight-tenths of the tensile unit.

It is at this point that the writer would ask, speaking from the viewpoint of a bridge designer, why reduce the compression value? Why not raise the tension value? Was not the 16 000-lb. unit chosen with a view to increased loads, and has not the test of years proven that railway bridges can carry, with absolute safety and without appreciable deterioration, much higher stresses than the equivalent of the 16 000 lb.? If the question of maximum loading is settled, the writer sees only extravagance in designing a steel railroad bridge for the tension unit stress of 16 000 lb., the usual allowance being made for impact and workmanship of the high standard generally required.

Neither experience with columns in structures, nor study of tests, convince the writer that there is any cause for alarm in the use of the 16 000-lb. compression constant in working formulas for steel columns, unless it be that one cannot depend on having columns properly proportioned and properly detailed. An analysis of most of the large columns which have been reported as showing unsatisfactory strength will show that the columns were defective in design, as compared with the requirements of good modern practice. The writer thinks that the principal trouble, if any, will be found in the column details, and that if the same attention is given to the concentric application of loads and to rivet connections as in tension members, ample provision is made for the full transmission of stress to all parts of members through details, and the ratios of width and length to thickness are kept down to the limits of conservative modern specifications, so that columns will have some body and not be "built of sheet iron," and there will be no failures nor cause for alarm with the 16 000-lb. compression constant in columns of ordinary size and construction.

It seems to the writer that, instead of being cut off on a horizontal line for very low values of $\frac{l}{r}$, say less than 20, a formula line should

theoretically rise abruptly to the compressive limit of the material at $\frac{l}{r} = 0$. This, of course, would make a complicated formula, and, as such short columns are unusual, it may be as well to omit this extra complication. It will be noted that Mr. Worcester omitted to plat values for columns having $\frac{l}{r} < 20$, although the series of tests he mentions includes a large number of such with values which would rise to the limits of his diagram. Mr. Carpenter.

In conclusion, the writer would state that he is opposed to a formula that is "chopped off" at the "long" end. Such a formula may be all right to design with, but it is "no good" for use in determining the strength of existing structures. A formula which best represents the true strength of a column, assuming its design is correct and its physical condition is up to the average, seems to be the proper one, and he does not know of any formula that fulfills this condition as well as the Gordon-Rankine formula, in the form:

$$\frac{C}{1 + \frac{1}{18\,000}\left(\frac{l}{r}\right)^2}$$

C. J. Tilden, Assoc. M. Am. Soc. C. E. (by letter).—The writer would like to call attention to one point, often overlooked, in regard to the use of the ratio, $\frac{l}{r}$, in the reductive term of practically all column formulas. In this ratio, l is the unsupported length of the column, and r its least radius of gyration, both measured in the same linear unit. The adoption of this ratio is equivalent to assuming that the flexure of a long column will take place in the direction of its least transverse dimension; that is, in a column of rectangular section, in a direction parallel to the shorter side of the rectangle; in a column of I-section, parallel to the flanges, etc. Mr. Tilden.

That this assumption is incorrect may be shown by investigating the resisting moment of a beam subjected to transverse loads, as the direction of the plane in which the loads act is changed.*

Take a concrete example: Let A B C D (Fig. 18) represent the cross-section of a rectangular prismatic beam, which may be subjected to transverse loads acting in any plane which contains the gravity axis, G, of the beam. The problem then becomes that of finding that direction of the plane of loads, M_X, which will cause the greatest fiber stress on the section for a given value of the bending moment. In

* A complete analysis of this question may be found in the paper by L. J. Johnson, M. Am. Soc. C. E., entitled "An Analysis of General Flexure in a Straight Bar of Uniform Cross-Section," *Transactions*, Am. Soc. C. E., Vol. LVI, p. 169 *et seq.* Professor Johnson's "S-polygon" furnishes a striking graphical proof of the facts demonstrated in this discussion.

Mr. Tilden. Fig. 18 this direction will be either M_{II} (the usual, but erroneous, assumption), or some position intermediate between M_I and M_{II}, as M_X.

For the direction, M_I, the maximum fiber stress of tension will occur on $C\ D$, and this will equal the compressive stress on $A\ B$. Similarly, for the direction, M_{II}, $A\ D$ will be in tension and $B\ C$ in compression. For any intermediate position, M_X, the maximum tensile stress will then occur at D, with an equal compressive stress at B. It is required to find that value of α, the angle between planes of M_X and M_{II}, which makes this stress an absolute maximum for any given value of M, the bending moment.

Resolve M_X into its two components, M_X sin. α and M_X cos. α, parallel, respectively, to the principal axes of the section. For M_X sin. α, the stress at D (and also at C) is a tension of

$$f' = \frac{6\ M_X \text{ sin. } \alpha}{b\ h^2}$$

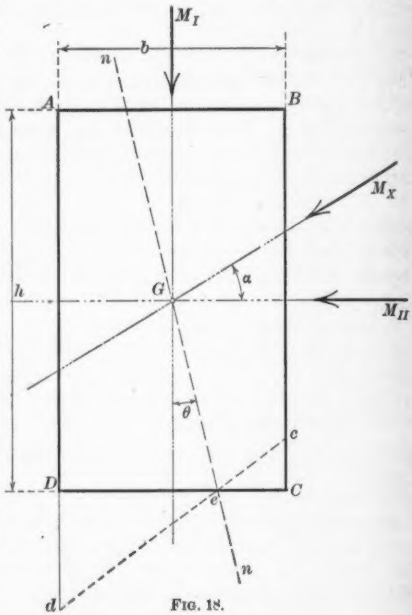


Fig. 18.

For M_X cos. α, the stress at D (and also at A) is a tension of

$$f'' = \frac{6\ M_X \text{ cos. } \alpha}{b^2\ h}$$

The resulting tension at D is, then,

$$f_a = f' + f'' = \frac{6\ M_X\ [b \text{ sin. } \alpha + h \text{ cos. } \alpha]}{b^2\ h^2}$$

obviously a function of α. To make f_a a maximum, M_X being a constant,

$$\frac{d}{d\ \alpha}\ (f_a) = 0 = \frac{6\ M_X}{b^2\ h^2}\ (b \text{ cos. } \alpha - h \text{ cos. } \alpha)$$

Whence

$$b \text{ cos. } \alpha = h \text{ sin. } \alpha$$

and

$$\frac{\text{sin. } \alpha}{\text{cos. } \alpha} = \text{tan. } \alpha = \frac{b}{h}$$

Mr. Tilden.

whence comes the important fact that the plane of least resistance to transverse forces, in the case of a rectangular section, is normal to the diagonal, $A\ C$.

Bending, however, will not take place in this plane; and, in order to find the direction of flexure, the neutral axis for this condition must be established. Assuming linear distribution of stress, and that the beam is subjected only to transverse forces (that is, the neutral axis passes through the center of gravity of the section), a second point on the neutral axis may be determined by finding the stress at C. This will be

$$f_c = \frac{6\ M_X\ (b \sin.\ \alpha - h \cos.\ \alpha)}{b^2\ h^2}$$

obviously, in this case, compressive. If this value is laid off to any convenient scale at $C\ c$, and the value of f_d, found above, is laid off to the same scale at $D\ d$, the straight line, $c\ d$, will cut $C\ D$ at e, a point of zero stress and therefore lying on the neutral axis, which is then established by drawing $G\ e$. Or, the distance $C\ e$ may be computed readily from the relation between f_c and f_d, and the tangent of the angle, θ, between n-n and M_1 determined in terms of b and h. This value is

$$\tan.\ \theta = \frac{b^3}{h^3}$$

The same principles may be applied to finding the weakest plane and corresponding neutral axis for any section. In Fig. 19, for example, is shown a column section built up of plates and angles. The plane of least resistance to bending is in the direction, M-M_1, and the neutral axis for this plane is n-n.

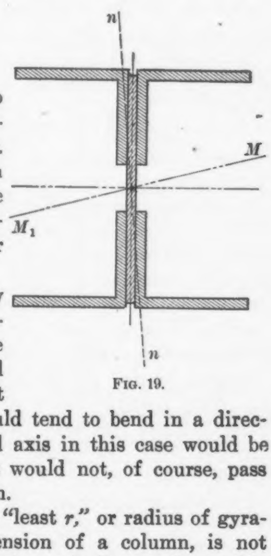


FIG. 19.

The foregoing analysis applies strictly to beams only. If, however, a member having either of the sections shown should be used as a strut, and subjected to an axial load which is carried to such a point that lateral flexure begins, the column would tend to bend in a direction normal to the line, n-n. The neutral axis in this case would be parallel to the direction of n-n, though it would not, of course, pass through the center of gravity of the section.

In view of the foregoing, therefore, the "least r," or radius of gyration parallel to the least transverse dimension of a column, is not strictly a measure of its greatest lateral weakness.* It may be readily

* G. S. Williams, M. Am. Soc. C. E., suggests that the variable in the reductive term should be made $\frac{l}{r \cos.\ \alpha}$, instead of $\frac{l}{r}$.

Mr. Tilden. shown by experiment that a long, thin strut, of rectangular or square section, flexed by an axial load, will not bend in the plane of its least dimension, unless forced to do so by restraint at the ends.

Empiricism, however, plays such an important part in the problem of the strength of columns, that it is perhaps not worth while to cavil at the mathematical inapplicability of a term the ultimate value of which, after all, rests on experiment. The author's formula is direct and conservative, and its graphical simplicity commends it highly; but the fact should not be forgotten that the reductive term of this formula, as of all its predecessors, is based on an erroneous assumption.

Mr. Jonson. ERNST F. JONSON, ASSOC. M. AM. SOC. C. E.—The speaker agrees with the author that the working unit loads now used for columns should be reduced, because they give a smaller factor of safety than that used for other structural members. On the other hand, he disagrees with the author's contention that an empirical formula is consistent with the best possible design, because such design pre-supposes an understanding of the nature of the column, in other words, a theory of columns. The speaker claims that the present high unit loads used for columns are due to empirical methods, or lack of proper theoretical treatment of the problem. He also claims that most cases of faulty column design, especially bad detailing, are due to the same cause; and, finally, that a large percentage of the column tests are more or less inconclusive because they were not made under the guidance of theory.

The theory of the column was developed by Euler, in the eighteenth century, and, because it was mathematically correct, was accepted as applicable to actual columns. When tests of columns were made, it was found, however, that they did not fulfill the predictions of pure theory. This experience placed the theory under suspicion, and called forth numerous empirical formulas. A more thorough study of the problem, however, revealed the fact that, while the theory was correct, it was the theory of a mathematically perfect column of a physically perfect material, and, therefore, had been wrongly applied when used in calculating actual columns without any allowance being made for imperfections of workmanship and material.

Every actual column contains innumerable imperfections, such as irregularities of form, lack of uniformity in the material, initial stresses, etc. All such imperfections have one thing in common, namely, that, in so far as they affect the equilibrium of the column, their essence is eccentricity, or divergence of the line of resistance from that of the load. No theory can take account of all the innumerable variations found in Nature; hence, in order that a problem may be treated theoretically, it must be more or less simplified. In the case of a column, this is done by substituting a small eccentricity of load-

ing for all actual eccentricities—an eccentricity which shall have the same effect on the equilibrium of the column as all the actual ones. Since the magnitude of this equivalent eccentricity is a function of the properties of the material and the workmanship, it must be determined, of course, by tests, like other physical properties. Mr. Jonson.

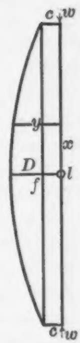


FIG. 20.

The essence of the theory of the column is as follows*: A column, unless absolutely perfect, can develop resistance only by bending, because it is subject to a bending moment which is the product of the load and the eccentricity. As the column bends, the load moment increases, but, at the same time, a resisting moment arises. When the latter moment becomes equal to the former, the column is in a state of stable equilibrium; hence it follows that the curve of flexure of a column is a curve of cosines, this curve, being the one in which the second differential coefficient, of which the resisting moment is a function, is proportional to the ordinate, of which the load moment is a function. The equation of the curve of flexure is, then,

$$y = f \cos. \frac{x}{r}\sqrt{\frac{w}{E}} \quad \text{(I)}$$

where y = ordinate, x = abcissa, $f = y$ maximum, or y for $x = 0$, r = radius of gyration, w = unit load, and E = modulus of elasticity (Fig. 20). Since the maximum external or load moment and the internal or resisting moment must be equal, it follows that

$$f = \frac{r^2 (p - w)}{a\, w} \quad \text{(II)}$$

where p = the maximum unit stress, and a = the distance from the neutral axis to the extreme point of section. Hence

$$y = \frac{r^2 (p - w)}{a\, w} \cos. \frac{x}{r}\sqrt{\frac{w}{E}} \quad \text{(III)}$$

If y is made equal to the equivalent eccentricity, c, then x becomes equal to half the length, l, of the pin-connected column.

$$c = \frac{r^2 (p - w)}{a\, w} \cos. \frac{l}{2\,r}\sqrt{\frac{w}{E}} \quad \text{(IV)}$$

This equation expresses the relation between length and unit load.

From Equation I a formula is obtained for calculating the value of c when the deflection, D, of the column is known. Let $y = c$, $f = D + c$, and $x = \frac{l}{2}$, and we have

$$c = D \frac{\cos. \frac{l}{2\,r}\sqrt{\frac{w}{E}}}{1 - \cos. \frac{l}{2\,r}\sqrt{\frac{w}{E}}} \quad \text{(V)}$$

*The development of this theory may be found in Bach's "Elasticität und Festigkeit."

Mr. Jonson. From this formula it may be seen that column tests are conclusive only when the unit load, the length, and the deflections in two directions at right angles to each other, are accurately determined. The first and third of these observations offer no special difficulty, but the length is seldom determined with sufficient exactness. The reason is that, for the purpose of a test, the length of a column is not the distance between its ends, but the distance between the two points on the axis of a column in which the bending moment is zero. In order to fix the location of these points, the bearings must be able to rotate without appreciable resistance around fixed centers. The best way to accomplish this would be by using knife-edge bearings of hard steel. If pin bearings have to be used, the pins should be of the least possible diameter, well supported, and the bearings in the ends of the column should be reinforced so thoroughly that they would not be strained beyond the elastic limit, even at the breaking point of the column. The deflection of the column should also be confined as much as possible to a direction perpendicular to the axis of the pin by making the column quite wide in the other direction. If columns of nearly equal stiffness in both directions are tested, ball and socket bearings should be used. In the case of the tests given by the author, it is not stated to what extent inconclusive and misleading tests have been excluded in plotting the diagram. Tests of columns with flat or fixed ends are inconclusive, because one cannot be sure of the absolute fixedness of the ends. More tests, and more refined tests, are needed for a satisfactory determination of the value of c. For the present, the speaker thinks that 0.25 r may be considered as a safe value for c.

The object of a factor of safety is to cover, with an ample margin, the maximum variation in each condition which affects the strength of a structural member. In the case of steel beams, a factor of safety of about $\frac{1}{2}$ is applied to the unit stress, and this also covers any possible variation in the load, because the result is the same as if, for instance, the factor, $\sqrt{2}$, is applied to the load and $\frac{l}{\sqrt{2}}$ to the unit stress. In columns, on the other hand, it makes a difference how the factor of safety is applied. Here it must be divided up on the unit load and the unit stress, that is, in order to get the same degree of safety as in a beam stressed to 16 000 lb., a factor of safety of 1.5 must be applied to the load, and a unit stress of 24 000 lb. must be used, and, besides, the value of the modulus of elasticity must be reduced to, say, 28 000 000 lb. Introducing these values into Equation V, the result is

$$0.25 = \frac{r\,(24\,000 - 1.5\,w)}{1.5\,a\,w} \cos. \frac{l\sqrt{w}}{8\,640\,r} \qquad \text{(VI)}$$

It is evident that this equation is too complicated for direct use in practice, and, therefore, it must be reduced to a curve. For this purpose, it may be written as follows: Mr. Jonson.

$$\frac{l}{r} = \frac{8\,640}{\sqrt{w}} \text{ arc cos. } \frac{0.375\, a\, w}{r\,(24\,000 - 1.5\, w)} \quad \text{(VII)}$$

THEORETICAL SAFE UNIT LOADS FOR STEEL COLUMNS.

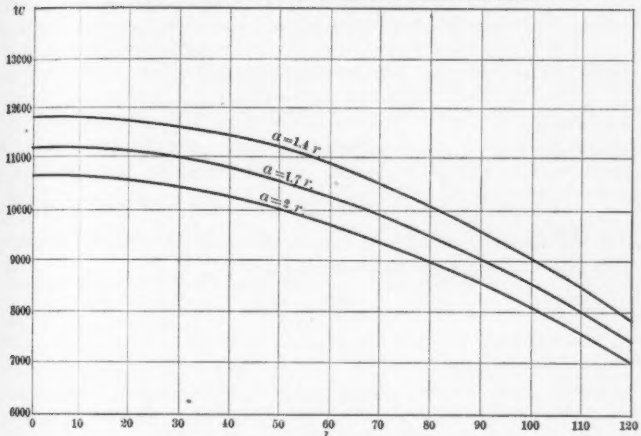


FIG. 21.

Curves for various values of a are given in Fig. 21. Although the factors of safety used for unit load and unit stress are the same as those commonly used for beams, and although those for the additional elements entering into the column problem are very moderate, the resulting unit loads are very much less than those commonly used. In other words, these latter unit loads do not give the same factors of safety as those given to other structural members. These curves also reveal another weak point in the present method of determining safe unit loads for columns, namely, that no account is taken of the relation of a to r.

The foregoing general theory of the column furnishes a basis from which a rational method of designing column details may be developed.

R. D. COOMBS, M. AM. SOC. C. E. (by letter).—Attention is called to the fact that in establishing a column formula, limited to values of $\frac{l}{r}$ between 0 and 120, no provision is made for certain classes of construction. Transmission towers and other light structures very frequently include compression members in which the ratios of $\frac{l}{r}$ are Mr. Coombs.

Mr. Coombs. greatly in excess of 120. It does not appear to be sufficient to characterize all such work as simply third-rate, and beyond the pale of proper specifications. Either the excessive values used for $\frac{l}{r}$ are scientifically wrong and their continued use a grave error, or a column formula reducing to 0 at 120 is equally wrong as a representation of fact.

It is claimed, by the builders of such towers, that their practice is based upon tests of full-sized structures, and, in the aggregate, a large number of such tests have been made, representing probably the largest series of full-sized tests of framed structures.

Without wishing to advocate excessive values of $\frac{l}{r}$, the writer would suggest that in competitive bidding, particularly when the financial resources of the purchaser are not great, the limiting value of $\frac{l}{r}$ is approximately 200. Table 12 has been compiled from miscellaneous designs covering a variety of work, presumably approved by different engineers throughout the country.

TABLE 12.

Height of transmission tower, in feet.	Member.	Section.	l	r	$\frac{l}{r}$
50	Horizontal strut.	One L $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{8}$ in.	138 in.	0.49 in.	281
50	Main column leg.	One L $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$ in.	138 "	0.69 "	200
62	Diagonal strut.	One L $3 \times 3 \times \frac{1}{8}$ in.	176 "	0.59 "	298
62	Main column leg.	One L $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$ in.	120 "	1.08 "	111
57	Horizontal strut.	One L $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{8}$ in.	138 "	0.49 "	281
57	Main column leg.	One L $4 \times 4 \times \frac{5}{16}$ in.	132 "	0.79 "	167
60	Strut.	One L $3 \times 3 \times \frac{1}{4}$ in.	140 "	0.59 "	237
60	Main column leg.	One L $4 \times 4 \times \frac{5}{16}$ in.	120 "	0.79 "	152
49	Main column leg.	One L $3 \times 3 \times \frac{3}{32}$ in.	84 "	0.59 "	142
49	Horizontal strut.	One L $2 \times 2 \times \frac{1}{8}$ in.	120 "	0.40 "	300
99	Horizontal strut.	One L $4 \times 4 \times \frac{5}{16}$ in.	180 "	0.79 "	228

Mr. Cain. WILLIAM CAIN, M. AM. SOC. C. E. (by letter).—In considering the merits of any new column formula, it is well to "take stock" of what we have that is sound and useful, and particularly to compare, side by side, correct theory with experimental data.

The "ideal column" is a prismatic, homogeneous column, without initial stress, having the resultant load applied at one end, in the direction of the straight axis, passing through the centers of gravity of the cross-sections. Although there are no ideal columns in practice, the theory pertaining to them is absolutely essential in order to understand fully the behavior of actual columns, or those which are not straight, not homogeneous as to material, strength, modulus, limit of elasticity, and perhaps with initial stress, besides eccentric application of the load.

Two diagrams, Figs. 22 and 23, are submitted; these are reproduced from the discussion by A. Marston, M. Am. Soc. C. E., on the writer's paper,* "Theory of the Ideal Column." In these diagrams, showing the results of Tetmajer's tests on columns, both of wrought iron and steel, the ends being pivoted, three curves are drawn. The upper curve, partly dotted, is drawn from Euler's formula, the full line is from Mr. Marston's formula, given on the figure, and the remaining dotted curve is from the parabolic formula of the late J. B. Johnson, M. Am. Soc. C. E. Mr. Cain.

In Euler's formula it is supposed that the column is "ideal" and that the load is applied without eccentricity.

First, with regard to Euler's formula: it has been shown by the writer that when $\frac{P}{A}$, or the average unit stress as given by Euler's formula, is greater than S_e, the elastic limit, the formula is inapplicable, even theoretically. When $\frac{P}{A} = S_e$, or is $< S_e$, corresponding to greater values of $\frac{l}{r}$ than when $\frac{P}{A} = S_e$, the formula gives the load at which bending just begins. Further, for a very small proportionate increase in the load, the column will fail from the stresses due to the considerable bending and the uniform compression. As a numerical illustration, a column, pivoted at the ends, 325 in. long, was assumed as built up of two 5-in. channels. The inch being the unit, $A = 3.9$, $I = 14.8$, and $E = 29\,000\,000$ lb. per sq. in. Euler's formula gives the load that causes incipient bending as 40 105 lb. It was computed, by an exact formula, that an increase of the load of only 5 lb., caused a deflection of 3.44 in. at the center, with a resultant stress on the most compressed fiber greater than the elastic limit. Finally, an increase of load of 2 or 3 lb. more would entail rupture, or a breaking in two of the column.

It is sometimes said that a load, as given by Euler's formula, only causes bending, and that, in the derivation of the formula, the uniform compression is neglected. In the writer's analysis, the uniform compression was considered from the start; also, the example shows that Euler's formula is practically a formula for rupture, since a few pounds added to the load (40 105 lb.) computed from it, leads to rupture.

It is admitted that Euler's formula has a limited application, but it is by no means useless to the practical man who has to erect poles in constructions, build derricks, etc.

A very important theorem follows from the foregoing: that for ideal columns, too short for Euler's formula to apply, no bending will occur, and the stress will be the same, and uniformly distributed, on

* *Transactions*, Am. Soc. C. E., Vol. XXXIX, pp. 109 and 111.

Mr. Cain.

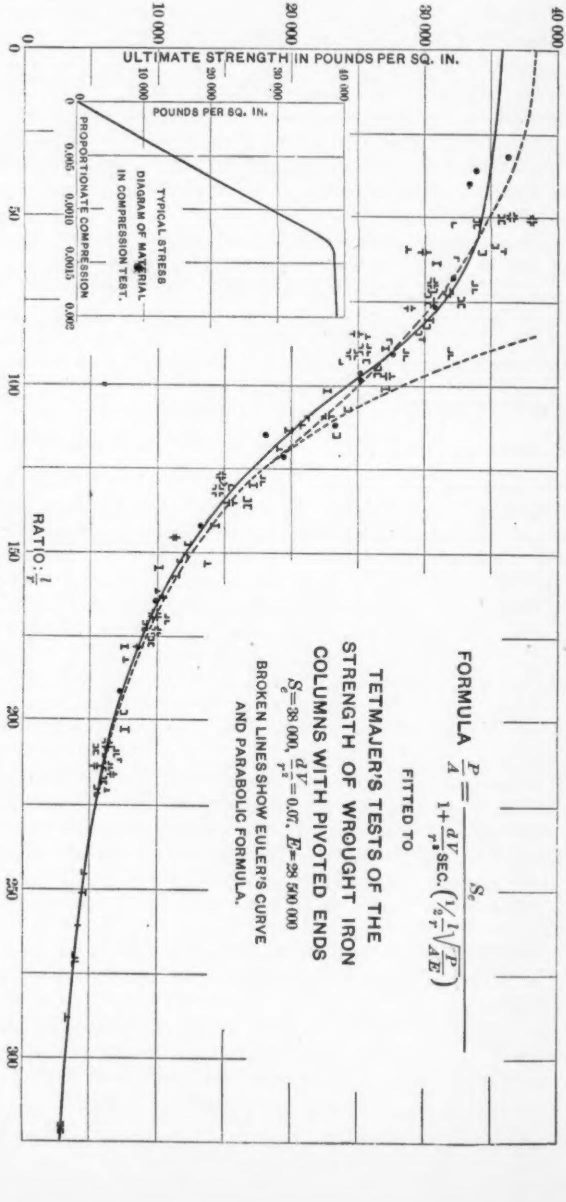


Fig. 22.

Mr. Cain.

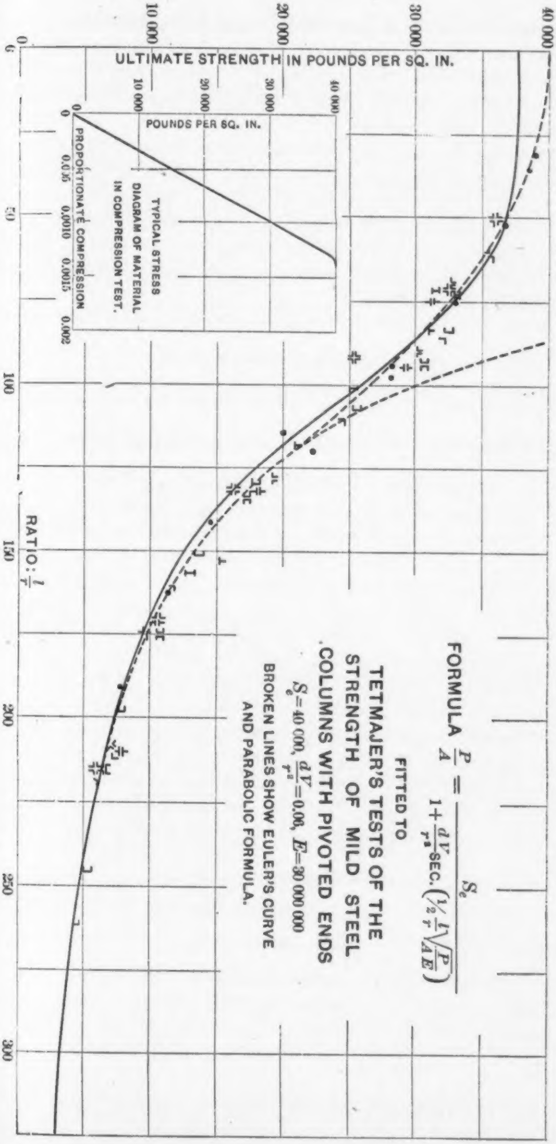


Fig. 23.

Mr. Cain. every cross-section, the elastic limit not being exceeded. A similar statement may be made in reference to long columns, to which Euler's formula is applicable, when the load is anything less than the formula gives. In these cases, the ideal column remains straight; there is no bending stress and no shear.

Next, consider the load, P, on the ideal column, to be placed at a distance, d, from the axis of the column. The resulting formula is given on Figs. 22 and 23. This formula is theoretically exact. In it, $A =$ the area of the cross-section, $r =$ the radius of gyration of the cross-section, about an axis through its center of gravity perpendicular to the plane of bending, and $V =$ the distance from this axis to the most compressed fiber.

To apply this formula to the actual column, Mr. Marston assumed, for wrought iron, $\frac{d\,V}{r^2} = 0.07$, and for steel, 0.06.

Now, it is true, as pointed out in the writer's paper (quoted previously), that when $\frac{d\,V}{r}$ is taken as constant, the ratio of d, the eccentricity, to the width of column, varies considerably for different shapes; but when it is considered that the eccentricity assumed for the actual column has to allow in a rough way for crookedness, lack of homogeneity of every kind, bad workmanship, initial stress, as well as the actual eccentricity experienced, the objection loses much of its weight. In practice, too, the line of force may be inclined to, or actually cross, the axis; so that the abnormalities to which the actual column is subjected are so manifold and various that it seems hopeless to deal with them all under the one head of eccentricity. The proof, however, is in the results. Mr. Marston's curves take a middle course through the whole set of plotted values for the wrought-iron columns, and nearly so for the steel columns. Here is seen a practical method of dealing with a truly rational formula, to give practical results for actual columns. No such definite conclusions as have been noted thus far can be reached by a simple observation of thousands of tests.

It may be remarked that the full curve in Figs. 22 and 23 practically coalesces with that corresponding to Euler's formula for $\frac{l}{r} = 225$, about, as should be the case.

Now, although the theoretical curves fit the experiments so well, the parabolic curves are just as good for practical results, as far as they extend; then Euler's formula or curve can be used for greater values of $\frac{l}{r}$.

A brief reference will now be made to another rational formula,

first given by the writer, in July, 1887,* and derived again on p. 120 of "Theory of the Ideal Column:" Mr. Cain.

$$\frac{P}{A} = \frac{S}{\left(1 + \frac{d\,V}{r^2}\right) + \frac{1}{8\,E}\left(S - \frac{P}{A}\right)\frac{l^2}{r^2}}.$$

In this formula, *d, V,* and *r* have the meaning previously given; *S* is the total fiber unit stress on the concave side of the column at mid-length. The column, of length *l,* is pivoted at the ends.

The only approximation used in deriving this formula was in assuming the neutral axis to be parabolic. The late Professor J. B. Johnson derived a similar formula in "Modern Framed Structures." When $d = 0$, an exact theoretical form is reached by replacing 8 by π^2.

Exactly as in the preceding case, to apply this formula to the actual column, $\frac{d\,V}{r^2}$ will have to be taken as constant. Unfortunately, to find the value of $\frac{P}{A}$ for a given column, a quadratic must be solved; hence the formula was not easily adaptable to computation and, consequently, was laid aside.

As Rankine's formula, which is of the form,

$$\frac{P}{A} = \frac{S}{1 + c\,\frac{l^2}{r^2}},$$

is sometimes spoken of as a rational formula for the actual column, it may be well to consider it briefly.†

As this formula does not suppose the load to be applied eccentrically, but does suppose bending, the latter must come from crookedness or lack of homogeneity; for, as has been seen, for values of $\frac{l}{r}$ less than pertain to Euler's formula, there can be no bending for the ideal column, and it is for just such lengths that Rankine's formula has been mainly used. Comparing it with the preceding formula when $d = 0$, it is seen that *c* cannot be a constant, since it is proportional to $\left(S - \frac{P}{A}\right)$, or the maximum unit stress at mid-length, on the concave side, due to flexure only. In fact, in the derivation of Rankine's formula, the assumption is made that the deflection varies as $\frac{l^2}{r} S$, whereas the very theory of beams to which reference is made shows that *S* must here be replaced by the maximum unit stress due to flexure only, or by $\left(S - \frac{P}{A}\right)$. This leads again to the preceding form of formula.

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* *Journal*, Franklin Institute.

† This part of the subject was discussed so thoroughly by Henry S. Prichard, M. Am. Soc. C. E., in *Engineering News* for May 6th, 1897, that an apology seems to be due for discussing it again.

Mr. Cain. The Rankine formula is thus irrational, and it is surprising that it should still be used in such problems as, for a given column and an assumed load, to compute S, assumed to be the total maximum fiber stress. Mr. Marston's formula, replacing S_e by S, should be used in such problems. As Rankine's formula can be made to fit the tests very well, it has been used extensively; but it must be relegated to the class of empirical formulas, like the parabolic, that fit the tests equally well and are more convenient to use.

The writer was very much impressed with the straight-line formulas of Thomas H. Johnson, M. Am. Soc. C. E., when they were first published, but is leaning now to the use of the parabolic formulas of the late Professor J. B. Johnson, the curves corresponding being made tangent to Euler's curve. In this way results are obtained which can be used in testing the strength of existing structures, as well as in designing. If, in designing, it is desired to exclude columns having the ratio, $\frac{l}{r}$, greater than 100 or 120, say, a simple clause to that effect in the specification should suffice.

It will be seen from this that the writer thinks that the author's intention can be carried out in a different way than in using the circular curve. If all the innumerable curves that have been proposed could be swept away and a curve be drawn by hand, steering a middle course between the plotted points (noting carefully, also, the lowest points), it would answer the purposes of the designer as well as a formula. For competitive designs, however, a formula is almost imperative. It should not give as large values, for very short columns—especially with riveted or butt ends—as the straight-line formulas. The imperfect "fixing" of the ends of some columns leads to such indefiniteness that it is customary, perhaps, to use the formula for hinged ends for all cases, though some allowance is often made for riveted ends or butt joints.

There is some indefiniteness, too, in the case of pin-end columns, on account of the friction of the pin; so that, for such columns, an eccentricity different from that used for pivoted ends would have to be assumed, perhaps, in applying the exact formulas of Mr. Marston. In all cases, the details of a built-up column must be designed carefully, for the details, rather than the length, are frequently the main factor in determining the strength of a column.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E. (by letter).—The discussion indicates a tendency on the part of the Profession to reduce the allowable compressive unit in columns, but there does not seem to be any unanimity as to the exact formula which should be adopted. The diagram, Fig. 24, includes, in full lines, all the definite curves proposed by those who have discussed the paper, and, in dotted lines, by way of comparison, the line adopted in the American Railway En-

gineering and Maintenance-of-Way Specification, together with that of the writer and the cut-off limit formerly advocated by Mr. Schneider. Mr. Worcester.

In plotting these, the writer has followed his interpretation of the discussion of Mr. Seaman as to the cut-off at the short end, and has assumed that neither Mr. Seaman nor Mr. Horton would carry the $\frac{l}{r}$ above 200. He would have been glad to have added Mr. Carpenter's, Mr. Jonson's and Professor Cain's curves, if they had given numerical values for the recommended units.

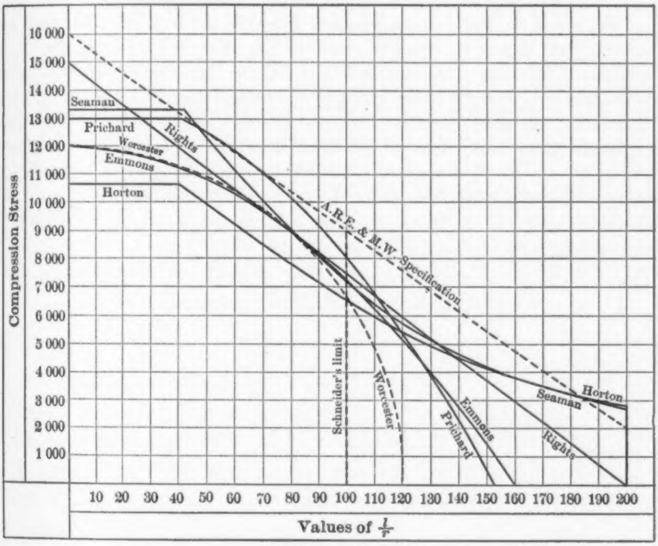

FIG. 24.

A study of this grouping shows a remarkable agreement of opinion in the vicinity of $\frac{l}{r} = 80$, but the divergence of opinion between $\frac{l}{r} = 100$ and 200 is deplorable. There is abundant evidence that columns have served their purpose with values of $\frac{l}{r}$ up to, and even exceeding, 200, but, for bridge structures, such slender members should be excluded. The writer does not believe that any of the advocates of the Rankine or Euler formulas would design bridges with the struts allowed by their specifications, and it does not seem to be scientific to make a specification which has to be used with reservations. While the theory of the ideal column is interesting, in the same way that astronomy is interesting to the mathematical mind, it has little bearing on practical

Mr. Worcester. designing when it is known that "the ideal column does not exist." In view of the deductions of Messrs. Prichard, Jonson and Cain, the writer hopes to see the Rankine form of curve eventually dropped, and one adopted which will be always concave toward the origin; whether it is circular, elliptical or parabolic is a minor consideration, as either one is preferable to a straight line or a Rankine or Euler shape.

The necessity of truncation at the short end seems to be generally admitted, and the exact unit at which this should occur seems to be conceded to be between 10 000 and 14 000 lb. The writer is well satisfied with his position in the middle between these limits.

Replying to the suggestions of Mr. Shearwood, Mr. Horton and others, that a proper consideration of the make-up and details of the column is necessary in connection with the allowable unit stress, the writer finds it difficult to see how any characteristic of the section, other than the radius of gyration, can be introduced, and, of course, it is apparent that this radius of gyration does not cover all the conditions of make-up. Is it not more rational, however, to specify that the details and distribution of metal shall be such that failure under test should occur by yielding of the column as a whole, and not by "wrinkling" or failure of component parts?

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1081

RECENT DEVELOPMENTS IN PNEUMATIC FOUNDATIONS FOR BUILDINGS.*

By D. A. Usina, Assoc. Am. Soc. C. E.

With Discussion by Messrs. F. W. Skinner, T. Kennard Thomson, Louis L. Brown, J. C. Meem, and D. A. Usina.

The purpose of this paper is to review briefly the recent and very interesting development in foundations of the class generally used for the high buildings being erected in the lower section of New York City. The earth there overlies a stratum of rock, the depth of which varies from 40 to 100 ft., and the enormous loads are carried most securely by concrete piers built with pneumatic caissons, and resting directly on the substratum of rock.

Prior State of the Art.—Prior to the present improvements, the conventional type of construction was as illustrated in Figs. 1 and 2. The working chamber was built with sides and roof of heavy timber or of sheet steel with stiffeners at suitable intervals. The coffer-dam was built up in successive sections (also of timber or stiffened steel), the horizontal joints being made by angles on the inside, and the walls being braced by transverse struts, where the shape and size demanded it. The shaft was of steel tubing fastened to the roof and at the several horizontal joints by outside angles.

* Presented at the meeting of April 15th, 1908.

As the structure was sunk, to bring the upper edge of each section of the coffer-dam near the ground level, a new section of coffer-dam and a new section of shafting were added, and the space between the coffer-dam and the shafting was filled with concrete. When bed-rock was reached, the working chamber and the shaft were also filled with concrete. The finished pier consisted of two entirely separate bodies of concrete—an inverted T-shaped portion bounded by the shafting and the roof and walls of the working chamber, and a ring-shaped portion surrounding the shaft and enclosed within the coffer-dam.

The surrounding shell, consisting of the coffer-dam and the sides of the working chamber, whether of timber or of steel, could only be considered a mould for the concrete and a curb or lining for holding back the earth during the sinking of the caisson. It could not be calculated as supporting any weight, but, on the contrary, was certain to rot or corrode in time, and leave a more or less free space around the pier. The shafting, and especially the roof, where the latter was of metal and was left in place, presented very serious possibilities. Their protection from corrosion depended on the care with which the concrete was rammed into contact with them. If either corroded to a substantial extent, it would produce a very large surface of weakness. The permanence of these important elements of the structure, therefore, depended on the care of workmen, who are not to be relied on for more care than is necessary at the moment. Furthermore, the angles at the several horizontal joints formed grooves in the concrete from 3 to 6 in. deep. Only under unusually favorable conditions could the shafting angles be calculated to act as supporting a share of the load in the ratio of their horizontal area to that of the complete cross-section of the pier; but the angles at the joints of the coffer-dam would not transmit any substantial pressure to the concrete below them, because the concrete would never be rammed under them sufficiently. The only transmission of pressure would be to the decaying or corroding walls, and the angles themselves would corrode in time. The greatest area upon which the bearing strain could be calculated correctly, therefore, was that within the inner edge of the angle-irons (*X*, Fig. 2), rather than that within the inner face of the coffer-dam (*Y*, Fig. 2). As a matter of fact, the latter standard was generally used, but the error was swallowed in the large factor of safety made necessary by the uncertainties of the problem.

Furthermore, the useless, and to some extent harmful, materials left in the ground, were very expensive parts of the structure.

There were thus two powerful incentives for the elimination of these materials from the finished structure, either by sinking the pier without them, or by withdrawing them after use. Nevertheless, there was a period of many years during which little or nothing was accomplished.

The recent activity in high building construction in New York City, however, making necessary a very extensive use of caissons of this type, has witnessed the substantial elimination of every material but concrete. The sinking of the coffer-dam and of a metal or timber roof for the working chamber, has been rendered unnecessary, and the steel shafting has been designed to permit its ready removal after it has served its purpose in the sinking of the caisson. These improvements have been put into practice in the foundations of the building for the United States Express Company, at the corner of Rector Street and Trinity Place; the New Trinity Building; the building for the United States Realty Company, at Broadway and Thames Street, and the Singer Building, on Broadway near Liberty Street.

Elimination of the Roof.—The most serious objection to caissons of the style described has been the existence of the roof, constituting a dividing plane across almost the entire cross-section. The objection to such a dividing plane was appreciated from the earliest use of pneumatic caissons. The late Theophilus E. Sickles, M. Am. Soc. C. E., in 1870, and John F. O'Rourke, M. Am. Soc. C. E., in 1898, proposed the removal of the roof after the sinking of the caisson and before the introduction of the concrete above the working chamber.

The Sickles caisson is shown in Figs. 3 and 4. The roof consisted of four segmental plates bolted to the under side of internal flanges of the casing and attached to each other by bolts passing through radial flanges on the under side. After sinking to the required depth, and sealing the cutting edge with a sufficient filling of concrete to prevent the entrance of water, the air was cut off and the roof removed by withdrawing the bolts passing through the several flanges. The caisson of the type shown had a high roof and no separate air-shaft supported upon the roof, as in the modern type, the coffer-dam or outer shell being made air-tight throughout its height. For a caisson of this type, the design of the roof was probably entirely satisfactory.

The O'Rourke caisson, Figs. 5 and 6, utilized a similar roof in half-round sections, but the roof was bolted on the top of the inward flange of the casing, and the flanges connecting the segments to each other were at the top. This would permit the filling of the working chamber with concrete clear up to the roof before removing the latter.

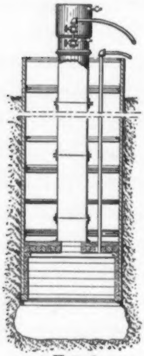

FIG. 1

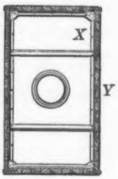

FIG. 2

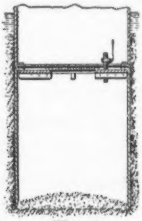

FIG. 3

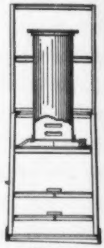

FIG. 5

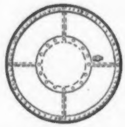

FIG. 4

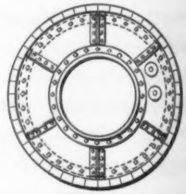

FIG. 6

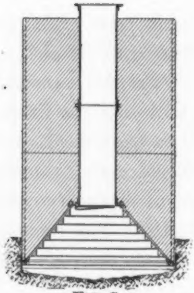

FIG. 7

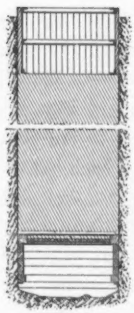

FIG. 8

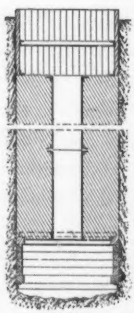

FIG. 9

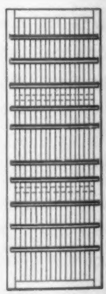

FIG. 10

The chief defect of these methods, however, appears in cases where, in order to get the requisite weight, the concrete is filled into the space above the roof during the sinking operation, as is usual in sinking

through earth for building foundations. In such operations it has been impossible to eliminate the roof of the working chamber until the introduction of a recent improvement which, at the same stroke, eliminated the coffer-dam which had previously passed for a necessary evil in sinking caissons in earth. The feasibility of the improvement was first demonstrated by sinking all the caissons for the building for the United States Express Company by this method, and at a substantial reduction in cost.

Elimination of the Coffer-dam.—There had been previously suggested, in 1904, the elimination of the coffer-dam and roof by sinking practically a solid pier of concrete, with only a central air-shaft and a working chamber hollowed out of the bottom. Fig. 7 gives a sufficient idea of the construction proposed. There was no distinction between different parts of the structure, except in so far as the lower portion of the concrete might be considered as the roof and side walls of the working chamber, and the concrete above this might be considered as the coffer-dam extending solidly from the surrounding earth to the shaft. It was proposed to build the whole of annular blocks of concrete laid one above another, or to form substantially a monolith by building up the structure *in situ* as fast as it was sunk. The difficulties in the way of moulding the concrete working chamber with suitably strong roof and sides and hardening it sufficiently in the short time available at the works then in hand prevented the utilization of this design, and, instead, the contractors adopted the design shown in Figs. 8, 9, and 10.

The working chamber was built of heavy timber, and across the top were laid angle-irons, a few inches below which was fastened a temporary flooring. The steel shafting was supported on this flooring, and a roof of concrete was moulded thereon to a substantial height, and of the same outside dimensions as the working chamber. The earth being excavated, and the chamber sunk to a sufficient depth, another section of concrete was added. The shafting was built up from time to time to maintain it above the concrete. After the first section of concrete was finished, the successive sections were moulded in place without interruption of the sinking operations; the excavation and the building up proceeding of course at the same ultimate rate, but quite independently of each other, and the coffer-dam, reduced to merely a mould for the concrete, being removed before the sinking of each concrete section.

In a previous design, it had been proposed to divide each section of the coffer-dam into flat units which might be readily transported and only united to each other when in place on the next lower section, this method having the further advantage of avoiding the necessity of breaking the air-pipes (see Fig. 1), which had been a cause of delay with the use of sections which were completed before being put in place; and such flat units were now found to be excellent moulding plates, only four being needed for each section of concrete, and excessive lengths being unobjectionable, because one might overlap the next at the corner.

The temporary flooring carried the concrete roof until the latter was hardened, and was removed before putting on the air pressure and the necessary lock. The angle cross-bars remained embedded in the concrete, transmitting its weight to the timber walls, although they were not necessary for the purpose after the concrete had hardened; and, in fact, after reaching a comparatively slight depth, the weight of the concrete was sustained by the skin friction and the air pressure, and added weights were necessary to force the caisson down. The cross-bars might have been designed and connected so as to permit their removal after the hardening of the concrete, if such removal had been thought of importance.

Only one accident occurred, and this demonstrated the advisability of using timber rather than concrete for the walls of the working chamber. The earth under one wall of the working chamber had been excavated previously to remove the footing of an old wall. When the first section of concrete had been moulded on this working chamber and the mould had been removed preparatory to sinking the concrete section, the old material replaced in the excavation allowed one side to settle so as to tilt the structure, and, before it could be righted, it fell over. The concrete was tied to the working chamber only by the crossing angles embedded in the base of the concrete, and swung bodily about the upper edge of a side wall of the working chamber, thus for a time putting its entire weight on this single wall. But the chamber was built so strongly that it was substantially uninjured, and the workmen in it at the time were unscathed. The accident, while indicating the necessity for greater precaution in building and sinking the first concrete section, demonstrated the practical excellence of the design.

When such a caisson was sunk to its final depth, there was no metal or timber roof to be removed. The cost of making first a sectional bolted roof, like that of Sickles or O'Rourke, and subsequently removing it, was saved; and, which is probably more important, the introduction of concrete above the working chamber did not have to await the sinking of the caisson. Its weight could be utilized in the sinking of the structure, and this weight, in caissons passing for a great depth through earth, is a very substantial consideration. It constituted probably the greatest of the series of advance steps under discussion.

Elimination of Shaft Lining.—The finished pier included, besides the concrete body, the cross-bars, which are a negligible consideration, being entirely embedded so as to prevent corrosion, and being of such slight cross-section as not to form cleavage planes in the concrete; and the steel shaft lining, which, at the very best, added not a pound to the load for which the pier might be safely designed, and, at the worst, might prove an element of weakness, and was certainly an element of substantial expense.

The progress of improvement in eliminating the shaft lining was the reverse of that in eliminating the roof. In the latter case, the idea was first advanced of making the roof removable after the caisson had been sunk; and the successful solution of the problem lay in avoiding the building of a true roof. In the case of the shaft lining, the first proposals endeavored to avoid its use entirely, but practical success came only with the idea of sinking the caisson with a shaft lining similar to those previously used, and removing the lining after sinking and before introducing the filling of concrete.

The first idea is shown in Fig. 11. A shaft lining of moulded concrete is shown. To avoid excessive loss by leakage of air through the concrete, it was proposed to coat the inner surface of the shaft lining with air-tight material, such as a paint containing lime. The difficulty of connecting the shaft lining to the air-lock with sufficient strength to resist the upward air pressure on the latter was to be obviated by long tie-rods extending from the lock to the lowest section of the shaft lining, as indicated in dotted lines. It was also proposed in this design to eliminate the lining entirely, merely coring the concrete body and coating the surface with paint, as above, the manner of fastening the air-lock not being specified.

The first successful attempt to eliminate the shaft lining, however, involved the use of a removable lining, which, while costing more than those of common design, is usable again and again indefinitely, and, in the long run, effects a great economy. The design used in sinking the caissons of the new Trinity addition, and the adjoining building of the United States Realty Company, is shown in Figs. 12, 13, 14, and 15. It was found that a comparatively small number of sections served for the sinking of many piers. There was no material loss of time involved in removing the sections and reassembling them for further use. In fact, the job was completed in much less than the previous record time for such work.

Figs. 12 and 13 show the shaft lining in place; Figs. 14 and 15 show the construction of one of the collapsible sections. Each section was composed of two approximately semicircular plates internally flanged for bolting to each other along one vertical edge, and a key interposed between the opposite edges of the plates. Internal flanges at the ends served for bolting successive sections to each other. Ladder rungs were arranged conveniently between the flanges of the key, and vertical guides were arranged just inside the line of the end flanges to guide the bucket past them. In some cases the tubing was made oblong in cross-section instead of circular. Packing was provided in all the joints, and this was the only part of the structure requiring renewal, it being cheaper to provide new packing for each re-use than to try to save the old.

Fig. 16 shows the finished pier, supposing the working chamber to be built of sheet steel. The dotted line indicates the joint between the concrete set up in sinking the pier and the filling introduced afterward.

Comparison with Concrete Piles.—Side by side with the progress in caisson work, recent years have seen a rapid improvement in the sinking or building of concrete piles in the earth. The first attempts to substitute concrete for timber or steel in piles contemplated the manufacture of the concrete piles above ground and the sinking of them by one or another of the methods used for timber or steel piles. But, at present, there are in the market several styles of concrete piles made by first forming the excavation and subsequently filling in the concrete. These methods permit the formation of piles of great depth and of theoretically unlimited diameter. Starting from widely-sepa-

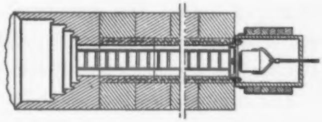
FIG. 11

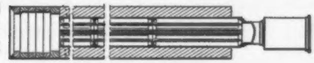
FIG. 12

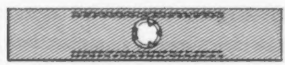
FIG. 13

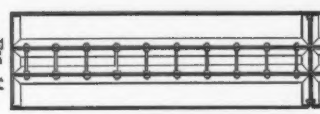
FIG. 14

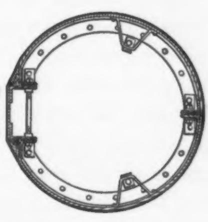
FIG. 15

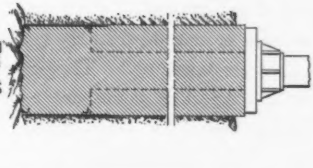
FIG. 16

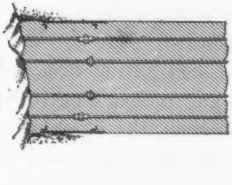
FIG. 17

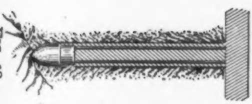
FIG. 18

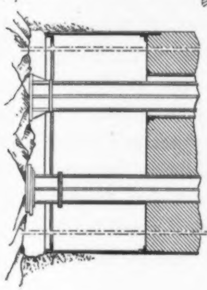
FIG. 19

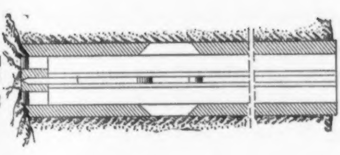
FIG. 20


FIG. 21

rated points, the two arts, caisson work and pile work, have constantly converged toward the same goal, a simple concrete column, bearing upon a rock or similar sub-foundation in the case of caissons and some piles, and supported by skin friction in the case of other piles.

The analogy has been carried even further by more recent improvements in which vertical reinforcing rods of steel, similar to those sometimes used in concrete piles, are embedded in the concrete of the pier. The base of such a pier is shown in vertical section in Fig. 17, and Fig. 18 shows a concrete pile similarly reinforced. The reinforcing rods in the pier should extend down to the rock sub-foundation, and are most easily introduced in that method of construction in which the roof of the working chamber is omitted, turn-buckles being introduced for putting the rods under stress before embedding them in concrete. The non-adjustable flange joints may be used for the rods which run through the shaft, and substantially the entire length of which may bear freely on the sub-foundation before the concrete is filled in about them.

Most Recent Modifications.—The steel rods in the foregoing designs merely reinforce the concrete. Should the concrete fail, or be designed or built so as to shift a substantial portion of the load to the rods, the latter would be unable to stand the strain. A recent design includes the introduction of columns of sufficient strength to carry a substantial load. In fact, they may be proportioned to carry all or the greater part of the load. Fig. 19 shows the caisson sunk to rock, and the columns in place, ready to be filled with concrete. The columns are of ordinary style, built up of Z-bars riveted to a central plate. One column is embedded in the concrete from the beginning, and is wedged up at its lower end. This column may be duplicated as often as desired. Another passes down through the shaft, and is properly supported before its embedment. The shaft lining may or may not be withdrawn, as desired.

Since it is possible to carry concrete piles in many cases to a rock sub-foundation, where they act as true columns, the idea has been conceived of substituting steel, with its immensely greater strength as a column, and surrounding it with concrete, which stiffens the column to some extent, but which performs the principal function of protecting the steel from corrosion. The finished pile or column is indicated in Figs. 20 and 21. The column is hollow, which serves to carry a

water-jet for sinking the column itself, and has a surrounding shell, which is afterward filled with concrete around and within the center of the column. The shell may be withdrawn as the concrete is introduced. The column may be shod at its lower end so as to secure a good bearing by ramming it down on the rock.

Invention is largely accidental, and its progress is apt to be most erratic. The writer has never observed a series of improvements progressing more logically and consistently in the same direction than those here considered. The engineering profession owes to Daniel E. Moran, M. Am. Soc. C. E., and John W. Doty, Assoc. M. Am. Soc. C. E., who conceived these improvements, and to the Foundation Company, by whom they were put into practice, a very large debt for the originality and progressive spirit with which they have met the demands of modern builders for economical methods of providing foundations of maximum bearing strength.

DISCUSSION.

Mr. Skinner. F. W. SKINNER, M. AM. SOC. C. E.—Mr. Usina has presented in a very interesting manner the recent developments of pneumatic-caisson work. The speaker agrees with the author in his description of some of the latest and most important developments; but as the first application of pneumatic-caisson work to foundations for buildings is scarcely more than fifteen years old, and as its development has included many changes from the original caissons, which were operated without essential deviation from the methods used in constructing submerged foundations for bridge piers, it may be excusable to call attention to a few other matters which seem to be important links in the chain of development, from the Manhattan Building to the United States Express Building, the City Investing Building, the Singer Building, and the Hudson Building, recently completed, all in New York City. There are only two or three instances where pneumatic caissons for buildings have been used outside of New York City.

The first pneumatic-caisson foundations for buildings—those sunk in 1893 for the Manhattan Building—were simply riveted-steel caissons, assembled partially at the site and sunk by the usual process. They were costly, and open to some objections from structural considerations. They were made excessively heavy, in order to carry the entire weight of the massive pier and its load, sustained, in the first place, by very heavy and deep transverse girders reinforced by knee-braces to the cutting edge, making a costly and complicated construction. Since that time this type has been entirely eliminated, an advantage largely due to the improvements described by the author.

One of the first important changes was the substitution of wooden walls and a wooden deck for steel in rectangular caissons, thus effecting a reduction in the cost and a greater reduction in the time. The rectangular caissons were built of solid courses of timber, sheeted and caulked inside and outside, and with crossed courses of timber, sheeted and caulked inside, for the deck.

The next considerable stride was made by the substitution of cylindrical caissons with wooden staves for rectangular ones. These were more easily made, handled, and sunk, and were more economical. John F. O'Rourke, M. Am. Soc. C. E., who has built a large number of difficult and important pneumatic-caisson foundations, is to be credited with that improvement. His courage, resourcefulness, and indomitable energy have been important elements in this field. To him is also due the credit for a simple method of controlling the escape of air from caissons. In sinking a caisson, it is often difficult to lower it after the cutting edge has been undermined a considerable distance. The side friction is sometimes so great that the weight in and on the

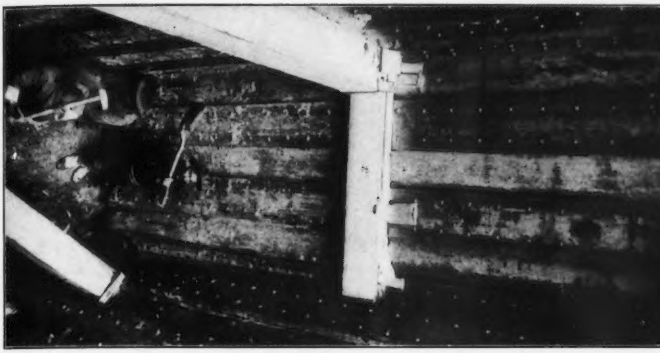

FIG. 1.—OLD-STYLE HEAVY STEEL SHEETING FOR FOUNDATION PITS.

FIG. 2.—DETACHABLE STUFFING-BOX ON HOIST ROPE.

FIG. 3.—DETACHABLE AIR-LOCK DOOR.

coffer-dam is inadequate to lower it, and the air pressure has to be diminished by letting it "blow out," as it is technically called. A blow-out is likely to be quite a critical operation, especially if the adjoining buildings are on quicksand. Thus the foundations might be jeopardized if material entered under the cutting edge in large quantities. Mr. O'Rourke simply bevelled the cutting edge, thus providing a high point which located the blow-out, and that was arranged to be on the safe side of the caisson. Mr. Skinner.

Still more important, however, and perhaps of greater value than any other single feature of the development of caissons, has been the improvement made by D. E. Moran, M. Am. Soc. C. E., in the air-lock for the passage of materials. The man-lock has been practically unchanged, but, originally, the air-lock, through which materials were removed from and introduced into the caisson, was like the man-lock, and it was necessary, in removing a bucket of spoil or introducing a bucket of concrete, to detach it from the hoisting tackle and handle it by separate apparatus inside the lock. That is wholly unnecessary now, but it was formerly impossible to keep the bucket in uninterrupted connection with the derrick.

After considerable experience with caissons, the upper door of the air-lock was made in two leaves, fitted on the joint line, with a detachable stuffing-box engaging the hoist line. The upper door can now be opened or closed regardless of the position of the line, which passes freely through it and need never be detached from the bucket. The stuffing-box permits the bucket to be handled almost as rapidly and as easily as in an open caisson, and with very small loss of air, and probably increases the rapidity of removing and entering material more than 100 per cent. Buckets can now make a round trip in and out of the caisson in less than 1 min., and previous to this improvement 2 or 3 min. were often required.

Another method of maintaining the attachment of the bucket to the hoisting line while passing through the air-lock was devised by Mr. O'Rourke, in the work for the Commercial Building. He made the top door of the air-lock detachable, and connected the stuffing-box permanently to it.

Another important development is the special arrangement by which the exterior caissons supporting the wall columns of several of the largest and most important buildings in New York City have been made to form a continuous water-tight wall, or, as it was termed in one of the first applications, a sort of dam enclosing on some or all sides the whole site of the building, and thus, theoretically and sometimes actually, avoiding the necessity of using pneumatic caissons for the interior piers. Having enclosed the building, a flow of quicksand from beneath adjoining buildings is prevented, thus allowing interior piers to be built in open excavations or coffer-dams.

Mr. Skinner. Considerable difficulty is found in making this construction, because there is a limit to the size of caissons which can well be sunk, and a length of about 30 ft. is a maximum for wall caissons supporting two columns or, possibly, three. In a building 100 ft. or more in length it takes several such caissons for the walls. Some clearance must be left between their ends, and that clearance may be from 4 in. to 12 or even 18 in., and its closure, and the permanent exclusion of ground-water and quicksand, in cases where the sand is very lively and the pressure is heavy, is a difficult problem, and has been met in several ways.

In the Commercial Cable Building, rectangular steel wall caissons were sunk a few inches apart. Pipes were jetted down between the adjoining ends, and after they reached the hardpan, 50 or 60 ft. below the surface of the street, they were filled with clay cartridges rammed by a piston or plunger operated by a pile-driver. The pipes were gradually raised, distributing the clay vertically, and such great pressure was developed by the ramming that it sheared the steel plates of the caisson itself, and very effectively excluded the water until the brick lining was built in the excavation.

This method was not in all respects satisfactory, and, not long after, Mr. O'Rourke devised a method of making a continuous concrete bond between adjoining caissons of the Stock Exchange and other buildings. As the caissons were sunk, semicircular recesses or wells were formed in the adjoining ends of both caissons and coffer-dams.

After the caissons were sunk and concreted, men entered and bolted the adjoining outer wooden walls together, removed the center part, between the bolts, and caulked the cut edges. The two wells were thus combined in a single one with a 4 by 5-ft. cross-section and an outline like that of a button-head rivet, which was filled with concrete, thus making a key and a bond between the caissons.

Later, various ingenious methods were used in bonding wall caissons together. In the Bank of the State of New York the wooden caissons were sunk about 8 in. apart, and one caisson was provided with a pair of 2-in. vertical timbers, 12 in. apart, recessed 3 in. into its thickened wall. These vertical ribs, which projected at first only very little beyond the face of the caisson, were equipped with 1-in. stud-bolts or set-screws projecting through the wall of the caisson and bearing against heavy steel yoke pieces secured by bolts to the wall of the caisson. After the caisson was sunk, the nuts on all the screws were operated to force the ribs out through the quicksand until their sharp cutting edges penetrated the wooden face of the adjoining caisson, and made a fairly tight and satisfactory joint, excluding the quicksand from the interior of the excavation.

Another method of connecting adjoining caissons was made somewhat more simply for the building at 42 Broadway, where the first

caisson was sunk with a couple of vertical 6 by 8-in. guide-timbers bolted to the end wall, the second caisson being located very close to these guide-timbers and provided with interlocking guide-timbers. Together, they formed a sort of tongued-and-grooved joint, effectually closing the space, and leaving a small core through which a pipe was jetted down, thus serving to scour out the sand down to rock and leave a space for the introduction of grout. Mr. Skinner.

Afterward, still another method was devised by the same contractors. A special sheet-pile was driven in the quicksand across the

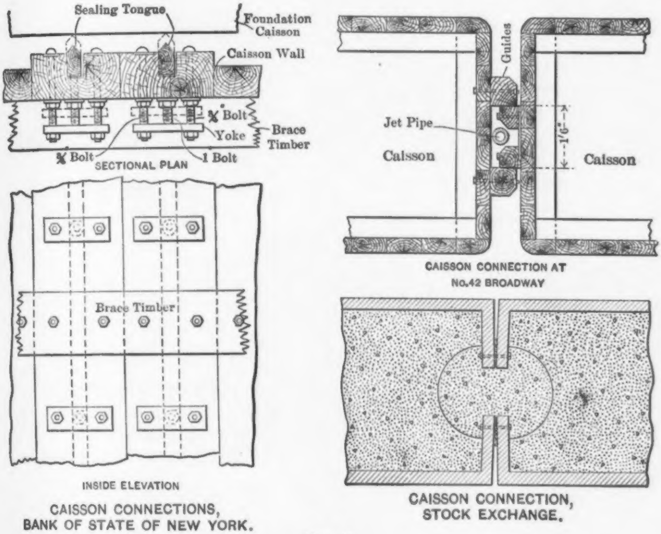

FIG. 22.

gap on both sides of the coffer-dams, protecting the narrow space between their ends, so that men could enter and excavate down to the deck of the caisson, below which the narrower space was cleared by post-hole diggers down to the cutting edge and the spaces concreted.

In the building for the Trust Company of America different methods were adopted for connecting adjoining wall-column piers. These piers were sunk about 4½ ft. apart by the pneumatic-caisson process before it was decided to make them continuous. Small intermediate wooden rectangular caissons, of the full depth of the piers, were sunk between them, and after they were landed on rock, alternate horizontal planks, A_1, A_2, A_3, A_4, A_5, A_6, etc., in the sides next the large caissons were successively removed, the sand scooped out, from

Mr. Skinner. the bottom up, and the spaces filled solid with concrete, after which the caisson itself was concreted, thus sealing the space between the large caissons or piers. The removal of the side pieces A_1, A_2, etc., was facilitated by the slots, B, B, B, etc., cut half through them when

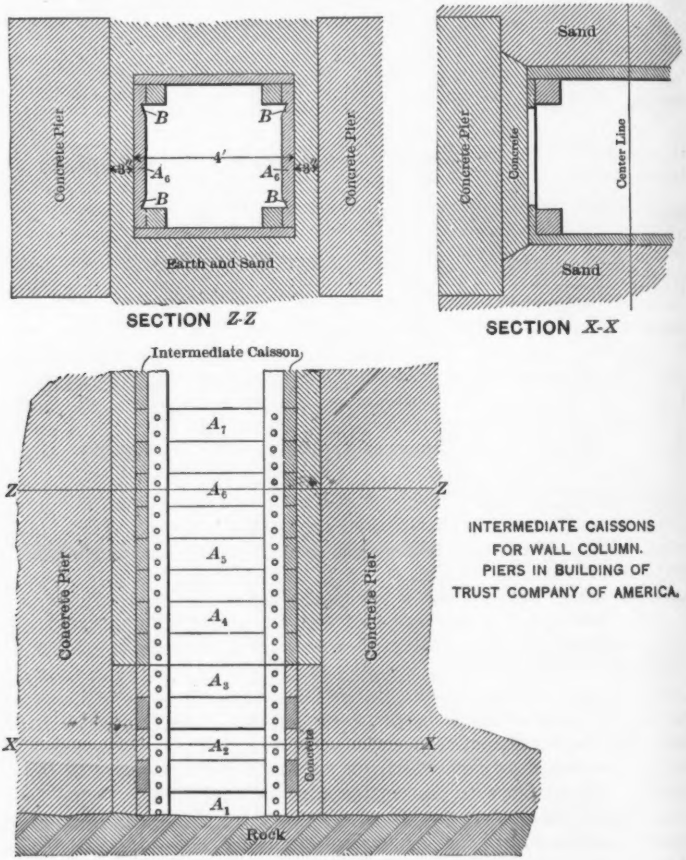

FIG. 23.

the caisson was built. Every other side piece was made without the slot, and was left in position to tie the caisson together.

Other pneumatic-caisson wall-column piers in the same building were sunk about 12 in. apart, with the intention of joining them after-

ward, and, to provide for this, semi-octagonal recesses 42 in. in diameter were cored in the concrete in the adjoining ends of the piers when the latter were made, and were closed on the outer faces by horizontal tongued-and-grooved side boards, *A*, *A*, *A*, making a well, *W*, in each Mr. Skinner.

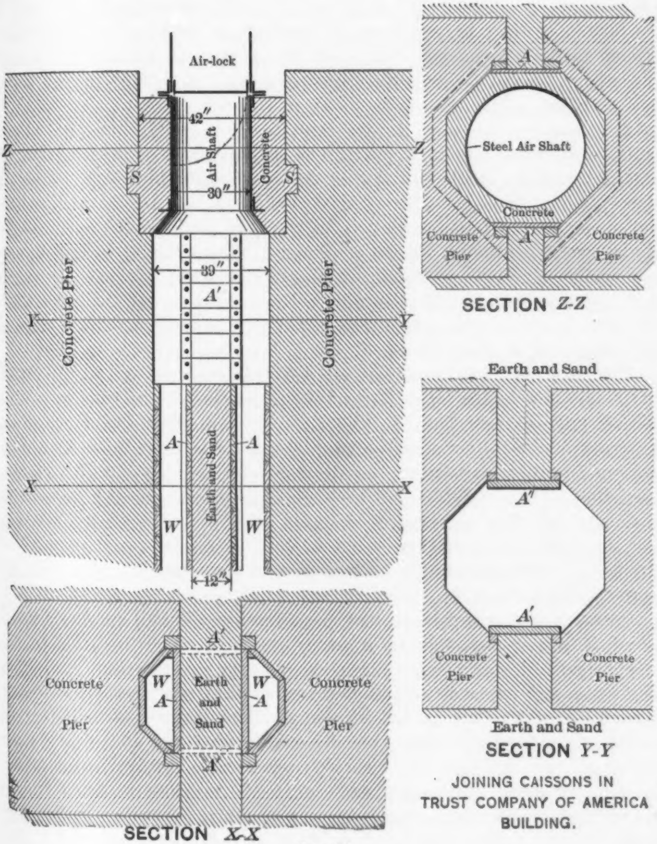

FIG. 24.

end of the pier. After the caissons were sunk and concreted, completing the piers, the earth and sand between them was excavated to the depth of 1 ft., and the first pair of boards, *A*, *A*, was taken out, cut and nailed on again in the position, A^1, A^1, at right angles to their first positions. This operation was repeated, thus completing the

Mr. Skinner. octagonal well between the two piers. When the well was deep enough, a short vertical section of a steel air-shaft cylinder was set in it and concreted, and an air-lock was assembled to it and pressure put on. Slots, *S, S,* in the pier concrete were filled with the shaft concrete, thus acting as keys to prevent the pressure from blowing out the shaft. The men then entered and continued the excavation and removal of the boards, *A, A.* In this way, the excavation between the piers was carried to the bottom and afterward concreted, thus making a positive and efficient bond between the piers, the first time that it had been accomplished under pneumatic pressure.

A minor improvement in pneumatic-caisson work, but one which contributes materially to economy and rapidity, is the substitution of 1 000-lb. castings, with connections for hoisting tackles, for the small pieces of pig iron formerly used to ballast caissons. An equivalent device is a heavy rectangular box in which is placed 1 000 or 2 000 lb. of pig iron, kentledge, or its equivalent. These boxes are compact, and easily handled and piled. Either form of ballast can be much more advantageously piled around the air-shaft or on the pier than pig iron, and can be handled very rapidly by a small hoisting engine, thus eliminating hand labor.

The reference to the connection between concrete piles and pneumatic caissons, in the latter part of the paper, is very interesting. The two are opposite extremes of difficult foundation work, but there is an important space intervening between them. The usefulness of a concrete pile terminates with the requirements for bearing strength greater than that of a single pile of such dimensions that it can be advantageously driven. The perfect concrete pile has not yet been devised, and, for loads of more than about 20 tons, there are few examples of anything except pneumatic caissons for many cases of pier foundations in soft ground.

Caissons are so expensive and excavation so difficult for piers from 3 to 5 ft. square that there is a very important gap to be filled in the construction of small piers having a capacity greater than that of a single pile and less than can be made economically with the pneumatic caisson. The speaker is not aware of any examples of satisfactory construction for such cases, but he knows that simple designs have been made, which appear to be entirely practicable and very economical, for sinking small concrete piers 3 or 4 ft. in diameter, in soft and wet ground, without the necessity of pneumatic-caisson or coffer-dam work.

On general principles, the reinforcement of either a pile or a pneumatic-caisson pier with steel rods in compression is to be avoided. Ordinarily, the pier or other foundation should be essentially a masonry structure; steel reinforcement should only be tolerated when a bending moment or flexure is unavoidable. For this reason the

speaker does not believe in the use of steel reinforcement for compression stresses in piles or in piers; he is aware that it has been used, and, further, he is aware that extremely high values have been permitted for steel used in this way in compression; but he has strong objections to it. Mr. Skinner.

Although the speaker wishes to express the utmost admiration for, and satisfaction with, the splendid work that has been done in pneumatic caissons, yet he thinks the tendency has sometimes been to overdo it. Pneumatic-caisson foundations are a form of construction essentially and inherently very costly, and, when not indispensable, very extravagant. Where equal security can be otherwise obtained, pneumatic caissons should not be used, although in many, perhaps in most, cases where they have been adopted their use has been unavoidable or justifiable. In some cases other forms of construction would have served equally well, and would have avoided excessive expense. The construction of the caisson is costly, and sinking it is costly. Elaborate and expensive plants have to be maintained for it, and in some cases the extreme cost could be obviated by the substitution of piers, sunk by open coffer-dam work and by other methods.

The use of improved steel sheet-piling will go a great way toward solving that problem, and reducing the cost of many difficult substructures. Up to the present time, steel sheet-piling, although it has been used in large quantities, has been of very heavy weight, has not afforded absolutely water-tight joints, has been subject to difficulty in driving, has cost from 75 cents to $1 per square foot and upward, and, therefore, has often been "thrown out of court," not only on account of its excessive first cost, but on account of uncertainty in driving.

Recent improvements in design and construction have very materially reduced the weight and cost of steel sheet-piling, and they insure absolutely water-tight joints, without caulking or silting. Such piling can also be driven with perfect protection, so that, no matter whether the driving be hard or easy, or whether or not there are moderate obstructions, the engagement of the piles and their perfect position can be assured when installed and in service. The piles, with ¼-in. webs, can bear a load of 100 lb. per sq. ft. with supports 6 ft. apart, and may be of any desired width, up to the limits of ordinary independent driving, say 24 in.

The most important elements of cost for steel sheet-piles are the joints, the spacing of which has heretofore been determined by the widths of standard rolled sections, and the practicable dimensions for driving. Both these considerations have been eliminated in recent improvements, by which steel sheeting can be installed, before excavation, in units of any width desired, thus reducing its cost almost one-half, without materially increasing the cost of driving, and by a method applicable in hard, soft, wet, or dry, soils. It can be used wherever

Mr. Skinner. it is possible to drive any other kind of sheet-piling, and provides for sheeting of any dimensions and any degree of strength and stiffness.

The significance of this fact is very important and far reaching; it means that, not only stiff sheeting can be installed more perfectly and cheaply than before, but that, where great permanent strength is not required, a continuous and perfectly water-tight steel surface, either temporary or permanent, can be installed, which has a minimum weight and, even with present facilities for manufacture—which can easily be materially improved—will cost less than wooden sheet-piling. This piling can be designed to have exactly the strength required, without excess of metal. If very thin webs suffice, they can be strengthened temporarily, to receive the reaction of braces and distribute pressure, by a facing of loose planks placed as the excavation progresses, and afterward removed, thus leaving only the minimum structure permanently engaged after the work is completed.

There is another point which has not been fully considered: In many cases it might be quite advantageous and entirely practicable to eliminate costly pneumatic-caisson work by enclosing one, two, or even three or four, sides of the building site by a continuous and perfect wall of steel sheet-piling. Suppose that 200 ft. of such a wall are necessary, and that it is required to be 30 ft. deep. It could be installed at a total cost of from $4 000 to $5 000, exclusive of salvage, and in many cases would not only protect the foundations of adjoining buildings from undermining and settlement—obviating the necessity for costly underpinning, in itself much more expensive than the total cost of the piling—but would also serve as a coffer-dam, excluding a large quantity of ground-water, so that the foundation piers could be constructed in open pits, much more rapidly and cheaply than with pneumatic caissons.

The pneumatic caisson is one of the most indispensable and important appliances for difficult substructure work; splendid courage, skill, and energy have been shown in the developments by which it has been brought to a high state of perfection and simplicity, enabling the erection of structures which would otherwise have been impossible. It will continue to be used advantageously in an increasing number of cases, but there will be other cases where the great cost of either pneumatic or rigid open caissons may be avoided, and perhaps structures built which would otherwise have been considered too costly, by the use of steel sheeting, which, besides the advantages mentioned, can be assembled before driving to form the walls of a complete caisson and driven sectionally in small units where it would be impossible to overcome the friction and resistance for the whole caisson at once, and where the driving effort can be concentrated on a small area of the structure, thus practically multiplying it greatly and also allowing for adjustment to conform to the rock surface and obstructions.

T. KENNARD THOMSON, M. AM. SOC. C. E.—The author is to be congratulated on giving such a clear demonstration of the art of caisson design as it existed in the early part of 1907. Mr. Thomson.

The design of pneumatic caissons has been completely revolutionized about four or five times in the last ten years, and the result is that what was the best two years ago was out of date last year, and last year's best is already out of date, notwithstanding the fact that there is no construction going on at present.

In trying to originate or improve, it is very common to revert unconsciously to original or antiquated types, and, after using these designs several times, the same objections which caused the original to be abandoned become apparent again. For example, the early caissons were built without coffer-dams on top, as the masonry piers were generally started on top of the massive wooden structures. To do this without any coffer-dam, of course, means that the masonry must be built as fast as the caisson sinks, as it always has to be kept above water. This means, first, delay in waiting for the masonry, and secondly, that the weight of masonry is often greater than required for sinking, which is dangerous.

It also means that the friction on the sides of the masonry from the surrounding material is often sufficient to break the fresh masonry joints; and again, stone masonry causes greater frictional resistance than plain greased boards. These are undoubtedly the reasons for giving up the attempt to build caissons without the use of coffer-dams.

The first pneumatic caissons under a sky-scraper were sunk for the Manhattan Life Building, at 66 Broadway, New York City, as Mr. Francis H. Kimball, the architect, was far-sighted enough to insist on a rock foundation for his building, and, though he was very severely criticised at the time, most people have become convinced of his good judgment, for pneumatic caissons are the only means by which sky-scraper foundations can be carried to good hardpan or bed-rock, in lower New York City, without serious danger to surrounding property. The only material above the hardpan is quicksand which runs like water, and anybody who attempts to hold back from 25 to 80 ft. of this stuff by sheeting, etc., will find the results disastrous to the pockets of the owners of the property. In fact, a great many thousands of dollars have already been lost through the experiments of novices, both by the novices themselves and their clients.

The caissons of the Manhattan Life Building were built of iron plates and angles, without coffer-dams, the brick piers being started on top of the deck.

Brick masonry has also been found to open at the joints, and allow the caissons and coffer-dams to separate. Brickwork and stone masonry, good in their day, have now, it is to be hoped, been entirely displaced by concrete for caisson work, both above and below the deck. Wet

Mr. Thomson. concrete requires little or no ramming or inspection after mixing, while masonry of stone or brick cannot be given sufficient inspection.

As timber costs much more than concrete, the less timber used, of course, seemed to show the greater saving, and it was thought feasible to replace the timber coffer-dams by temporary forms, as described by the author.

It was soon found, however, that the apparent saving was very deceptive, for, in the first place, the concrete has to be allowed from 24 to 48 hours to set, and the result is that the sinking has to be interrupted several times, for several days, the men being put to work elsewhere. To change the men from one caisson to another in the middle of a shift is, of course, a waste of time; and to keep the air pressure on a caisson about twice as long as would be necessary with coffer-dams is obviously expensive, as well as dangerous, for when no one is working in the air chamber the gauge tender is likely to, and often has, become careless, allowing the pressure to increase or decrease, with disastrous results, especially to adjoining property.

Another reason is that, for economical sinking, it is necessary that the penetration should be gradual but continuous; plunging a couple of feet at a time and then waiting for several hours gives the quicksand a chance to flow against the sides of the caisson and then adhere to it. Naturally, if the caisson stands still for a couple of days or longer, this trouble is much increased, and it is quite difficult to start it again, and often requires a great deal of additional weight, usually in the form of pig iron, thus increasing the expense.

A minor objection to the "forms" is that they are usually connected by iron angles bolted together, and in New York City the Unions insist upon this being done by the Iron Union, thus making it necessary to keep a high-priced gang for unskilled work. Therefore, the probabilities are that timber coffer-dams, from the top of the caisson to the surface of the ground, will be retained.

The omission of the timber or steel roof is often a very decided saving; but in some cases it would not pay and in others it would be dangerous, especially where it would make the caisson too heavy to be handled to advantage. If the caisson with its load is too light, of course, it will not go down, whereas if it is too heavy it will penetrate more rapidly than desired, and, as a matter of fact, the cutting edge has frequently been forced into the ground until the entire working chamber has been filled with earth, etc. This, of course, would kill any of the men who were unable to escape, and would make the resumption of work quite tedious, as it would be necessary to remove enough material to make room for the men and the bucket.

As for the steel shafting, which is expensive, it is often economical to omit or remove this; but here, again, the question of extra labor and time will often overcome the advantages, and, in the case of a

5-ft. circular caisson with a 3-ft. shaft, it can easily be understood that the omission of the steel shaft would be attended with considerable risk, as experience has proved, where the caisson and concrete have broken apart—an accident almost impossible to rectify when the caisson is from 20 to 40 ft. in the ground. Mr. Thomson.

There have been several cases where this has occurred; and what assurance is there that similar accidents have not occurred without being detected?

In short, even the most experienced men are often compelled to learn by experiment, while novices in foundation work must cultivate a very profound respect for earth and water pressures, or pay the penalty, which on one contract known to the speaker amounted to about $75 000.

Louis L. Brown, M. Am. Soc. C. E.—There is little that can be added to this paper, especially considering the remarks made by Mr. F. W. Skinner and Mr. T. K. Thomson, both of whom are very conversant with the subject. Mr. Brown.

From the standpoint of one whose experience has been closely connected with the actual execution of work of this class, the fact of the recent improvements in methods cannot be given too much importance.

While it is true, as stated by Mr. Thomson, that there have been a great many improvements and changes in a very short period of time, and that sometimes new ideas are tried and found to be unsatisfactory, still, to make sure on such points, actual trials are necessary.

One company in New York City has tried and is continually trying new methods, for the purpose of cheapening and bettering caisson construction, and out of the many trials and investigations which this company has made have come some of the recent improvements such as Mr. Usina has illustrated.

Not only have the cost of the work and the excellence of construction been considered, but also the safety of the workers. There is now in use a shaft which, besides having the features described by Mr. Usina, is of oval cross-section, with space at one end for the passage of the bucket and at the other for a ladderway to permit the men to pass in and out without the risk of being hit by a descending bucket. This has averted many accidents.

J. C. Meem, M. Am. Soc. C. E.—There is one type of caisson which the author did not note in his paper, and, as it may be of general interest, it is here described. Mr. Meem.

Some six months previous to the opening of the Interborough Rapid Transit subway extension to Brooklyn, now known as the Battery Tunnel, a contract was let to the Cranford Company, of Brooklyn, for the building of a large shaft at Willow Place, on Joralemon Street, to serve the several purposes of an emergency exit, a duct manhole, and a ventilating shaft.

Mr. Meem. The depth at this point is about 64 ft. to the outside bottoms of the tubes, which are practically circular, cast-iron-exterior, concrete-lined tubes of about 17 ft. external diameter. The water level at this point is slightly below the tops of the tubes. The required opening of the main shaft (irrespective of chambers and appurtenances) was about 54 by 19 ft., outside measurements, and about 66½ ft. deep.

After making various studies of methods, the contractors decided to do this work by what may be called the "place caisson" method, as indicated in the sketch, Fig. 25. This consisted essentially in excavating to water level by ordinary methods and in erecting there the upper walls and roof of a caisson, then finishing the remainder of the excavation under air pressure. The upper part of the shaft was first partially decked over, then excavated to the water level, the sides being protected by 2-in. sheeting set in horizontally and braced vertically and across. Between the tubes, adjacent to the sides and across the ends of the shaft, U. S. steel sheet-piling was driven to a penetration of from 12 to 15 ft., or as far as possible without jetting; and between the steel piling 2-in. sheeting was driven to the tops of the tubes, the bottom of each plank being cut to fit as closely as could be done by rough calculation. Waling pieces were then bolted to this piling, and the tops were cut off to a level bearing, and to these waling pieces was bolted a roof of 12 by 12-in. timbers. The roof, and, as far as possible, the sides, were then air-proofed and internally braced, and material and man-locks of the vertical type, together with air and other pipes, were set in through the roof. The whole shaft was then backfilled above the roof to a sufficient depth to counteract a possible maximum air pressure of about 8 lb.

Air under pressure was then introduced, and the sheeting and excavation proceeded until the bottom of the shaft had been reached, when the bottom and sides were water-proofed with brick and mastic. This water-proofing was sealed to the outside of the tubes by plate-iron collars tap-bolted to them. The regular construction of concrete and steel, as designed by the Public Service Commission, was then built inside the caisson, and when this construction had reached the normal water level the air pressure was taken off and the roof of the caisson removed. The tubes inside the limits of the shaft were then cut out and removed, leaving the shaft entirely free and with openings into each end of both tubes, giving four openings in all.

As noted, the inside of the caisson below the bottoms of the driven sheet-piling was sheeted with 2-in. planking set in horizontally and air-proofed as far as possible. On reaching a point near the bottom, however, it was found best, as far as possible, to drive a row of "toe" sheeting around the sides of the caisson on the inside, keeping the bottom of the sheeting well below the external bottom of the shaft.

Owing to the fact that the ground had been shaken up consider-

Mr. Meem.

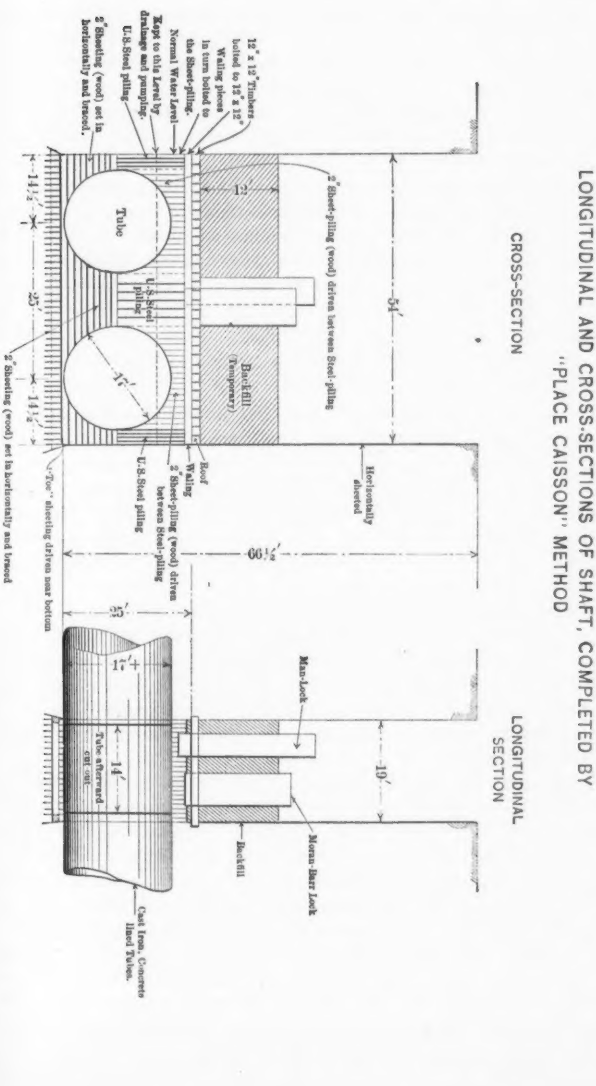

FIG. 25.

Mr. Meem. ably during the original construction of the tunnel, and the further difficulty of making a tight joint between the tubes and the sheeting, a larger quantity of air was required than was expected, and there was great difficulty in keeping the water down for the last 2 or 3 ft. of the excavation. Outside of this, there were no difficulties other than those anticipated by the contractors.

By using this method, the Interborough Company was enabled to have the full possession of the interiors of the tubes for the work of laying the track, lining the tubes, and other work incidental to the preparation for operation, all of this work being carried on simultaneously with the shaft work on the outside of and around the tubes. In spite of the loss of air, no settlement whatever was noticed in adjoining buildings, some of the foundations of which were within 5 ft. of the exterior lines of the excavation. This was believed to be due largely to the fact that especial care was taken in setting in and bracing the sheeting of the upper shaft. The work was done under the direction of the Public Service Commission, and for the Interborough Rapid Transit Railroad Company.

Mr. Usina. D. A. Usina, Assoc. Am. Soc. C. E. (by letter).—The notes by Mr. Skinner, covering all the modern improvements in pneumatic foundation work, are a very valuable contribution to the literature of this art. In fact, it can hardly be said that there is any adequate printed description of the methods used in work of this class. The standard authorities on tunneling and foundation work usually confine themselves to the description of caissons which, at least for building foundations, are extremely antiquated.

The shifting from steel to wood and from wood back to steel seems to be more due to economy, as the relative prices shift, rather than to superiority of design, but the air-lock improvements referred to, by which the removal of the materials was made a continuous operation instead of an interrupted one, and the several suggestions for effecting tight joints between adjacent piers, have been highly advantageous as well as ingenious.

It is generally agreed that pneumatic caisson foundations are so costly as to be usable only where they are indispensable. Improved methods and machinery have somewhat reduced the cost, but they still remain by far the most expensive foundations. On the other hand, the buildings which are supported by such foundations in New York City represent enormous investments, and engineers are well advised in holding it indispensable that such buildings should be supported upon bed rock. The simple designs referred to for sinking small concrete piers 3 or 4 ft. in diameter without the necessity of pneumatic caisson or coffer-dam work will surely find a wide use when they are reduced to practical form. Schemes proposed for this sort of work, however, are not very promising.

Mr. Thomson is disposed to smile at the improvements which he sees "advancing" in a circle. But history does not repeat itself any more accurately in engineering than in any other science. The complete displacement of stonework and brickwork by concrete, the improvements which the past fifteen years have witnessed in the manufacture of cement and concrete, the character of the buildings supported, and the conditions under which the work is carried on, have all contributed to the feasibility or necessity of improvements resembling in general, but differing in important points from, the experiments of years ago which were abandoned because they were neither feasible nor necessary. The progress of improvement obeys the law, not of a circle but rather of a spiral, going forward and backward, but deflected always outward by new conditions. Mr. Usina.

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TRANSACTIONS

Paper No. 1082

EFFECT OF EARTHQUAKE SHOCK ON HIGH BUILDINGS.*

By R. S. Chew, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Guy B. Waite, E. G. Walker, Eugene W. Stern, and R. S. Chew.

In submitting this paper, the writer is well aware that he is dealing with a force that can be measured only by the resistance encountered; and it was simply with a view of determining the nature of the stresses induced in structures by a shock, such as that in San Francisco on April 18th, 1906, that the following was undertaken.

All realize that, with a possible exception, the steel-framed structures in San Francisco stood this shock. This fact has promoted confidence, and has satisfied architects and owners that such is the safe type of building for the Pacific Coast. The engineer, however, cannot be satisfied until he ascertains just how a disturbance of this nature affects high buildings.

The effect of an earthquake is to produce a complex movement in the crust of the earth. This movement is a wave motion accompanied by more or less twisting. This twisting or torsional effect is small, and affects only the first tier of columns. The length of the wave is very long, so that the vertical movement is small, and, for a structure with a well-designed foundation, may be neglected. The effect of the shock, then, is from the horizontal motion, which is a rapid oscillation.

*Presented at the meeting of March 4th, 1908.

From mechanics, it is known that:

$$\text{Force} = \text{Mass acceleration, or } F = M\,a = \frac{W\,a}{g} \ldots\ldots\ldots\ldots (1)$$

or, the force of the earthquake on any structure is the mass of the structure into the acceleration produced.

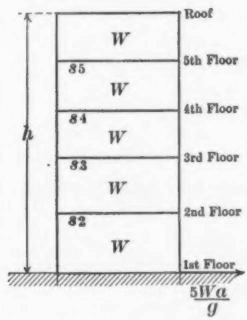


FIG. 1.

Imagine the structure represented by Fig. 1 to be built of a perfectly rigid material, and that W is the weight of each story; then, as the foundation takes up the movement of the earth, it endeavors to set the structure in motion. The inertia of the building resists, and calls into play the force, $F = \frac{5\,W\,a}{g}$. The shearing stresses at each floor are:

$$S_2 = 4\,\frac{W\,a}{g}$$

$$S_3 = 3\,\frac{W\,a}{g}$$

$$S_4 = 2\,\frac{W\,a}{g} \text{ etc.}$$

The maximum bending moment $= \frac{5\,W\,a}{g} \times \frac{h}{\;}$. If these shears and bending moments could be developed, the building would follow the movement as a whole. There are, however, no perfectly rigid materials, so that, under the action of a force, there would be deformation which, as will be seen later, is different in different types of buildings.

Consider first a structure which has no wind bracing. By reference to Fig. 2 it will be seen that each story weighs W, and that, there-

fore, the resistance that each story would offer to having set up in an acceleration, a, would be $\frac{W a}{g}$.

The building, under this action of forces, is a beam cantilever under a uniform load so that:

$$\text{Moment of inertia at section } 1-1 = I_1$$
$$\text{" \quad " \quad " \quad " \quad " } \quad 2-2 = I_5$$

assuming $I = \frac{I_1 + I_5}{2}$ then, approximately,

$$\varDelta = \frac{5\ W a}{g} \times \frac{h^3}{8\ E\ I} \quad \text{......................} (2)$$

It will be noticed that $\varDelta$, in Equation 2, does not represent the displacement due to the shock, and may be greater or less than this.

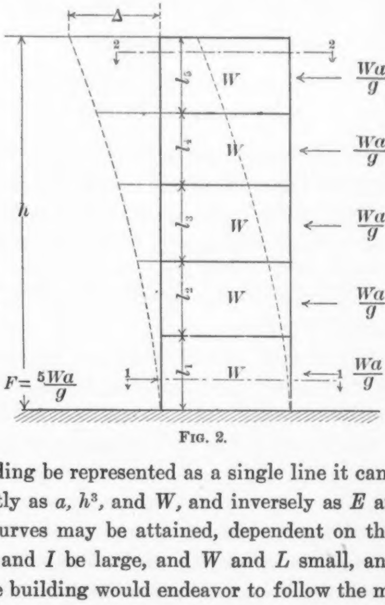


FIG. 2.

If the building be represented as a single line it can be seen that, as $\varDelta$ varies directly as a, h^3, and W, and inversely as E and I, that either of the three curves may be attained, dependent on these variables.

If now, E and I be large, and W and L small, and the oscillation rapid, then the building would endeavor to follow the movement closely, in which case the curves produced by the oscillation would produce a wave motion in the structure, as shown approximately by Fig. 4.

The writer believes this to be the effect of the late shock as felt on the majority of low structures. He identifies it as the effect produced

on the three-story apartment house which he was in on that memorable morning. It was quite different from the effect felt in the fifth story of a six-story steel-frame structure during a later shock. In the latter case, there was a decided swinging movement of the building.

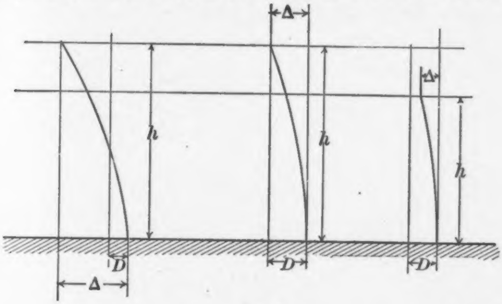


FIG. 3.

Referring to Fig. 1, one may note the high shearing stresses produced. These are, of course, the same in Fig. 2. The bending moment is a maximum at the foot of the column, and equals:

$$\frac{5\,W\,a}{g}\,\frac{h}{2} + 5\,W\,\frac{\Delta}{2} = \frac{5\,W}{2}\left(\frac{a\,h}{g} + \Delta\right) \ldots\ldots\ldots\ldots (3)$$

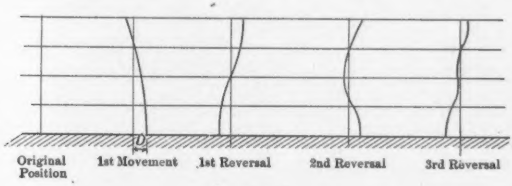


FIG. 4.

It can be seen at a glance that this type of construction is not adapted to resist any shocks except those of very small displacement; and, even in these, the buildings will fail in detail. For instance: It was noted, after the shock of April 18th, that in a number of cases the connection of beams to columns had failed by the rivets shearing off. Reference to Fig. 5 will explain the condition. By referring to the curves, it will be seen that during the vibration the column bent, throwing the

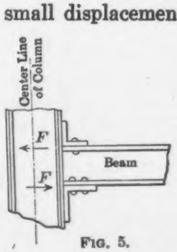


FIG. 5.

couple, $F F$, into action, and the rivets, not being sufficient to stand this, failed. The writer has also noted several buildings of this type in which there is a decided crack in the brickwork following the column, which tends to substantiate this theory.

A wind-braced building will act a little differently from the foregoing, due to the fact that the point of contraflexure in the columns is fixed by the bracing, so that the building in part will follow the movement.

Let D = the horizontal displacement,

t = the time for said horizontal displacement,

a = the acceleration $= \frac{D}{t^2}$,

g = the effect of gravity;

then, the force exerted on the building $= F = \frac{W a}{g} = \frac{W D}{g t^2}$, where W equals the weight of the building.

If, in Fig. 6, it be assumed that the horizontal girders are stiff enough to fix the columns at the knees, then the effect on the building by the movement, D, is as shown.

$D' = \lambda_1 + \lambda_2 \ldots\ldots + \lambda_5 = 2\ (\Delta_1 + \Delta_2 \ldots\ldots + \Delta_5)$ = the deformation in building.

W_5 = the weight of the building above and including the first floor.

W_4 = the weight of the building above and including the second floor.

W_3 = the weight of the building above and including the third floor.

From mechanics, it is known that:

$$\Delta = \frac{F l^3}{3 E I} = \frac{W D}{g t^2} \frac{l^3}{3 E I} \ldots\ldots\ldots\ldots (4)$$

$$\Delta_5 = \frac{W_5 l^3_5 D}{3 E I_5 g t^2}$$

$$\Delta_4 = \frac{W_4 l^3_4 D}{3 E I_4 g t^2}, \text{ etc.}$$

$$\text{Therefore } D' = \frac{2 D}{3 E g t^2} \left(\frac{W_5 l_5}{I_5} + \frac{W_4 l_4}{I_4} + \frac{W_3 l_3}{I_3}, \text{etc.}\right) \ldots\ldots (5)$$

or, if $D = D'$,

$$\text{therefore } 1 = \frac{2}{3 E g t^2} \left(\frac{W_5 l_5}{I_5} + \frac{W_4 l_4}{I_4}, \text{ etc.}\right) \ldots\ldots\ldots (6)$$

The bending moment is a maximum at the base, and

$$= F\, l_5 + W_5\, \Delta_5$$

$$= \frac{W_5\, D\, l_5}{g\, t^2} + \frac{W_5^2\, l_5\, D}{3\, E\, I_5\, g\, t^2}$$

$$= \frac{W_5\, D\, l_5}{g\, t^2}\left(1 + \frac{W_5\, l^2_5}{3\, E\, I}\right) \quad \ldots\ldots\ldots\ldots\ldots(7)$$

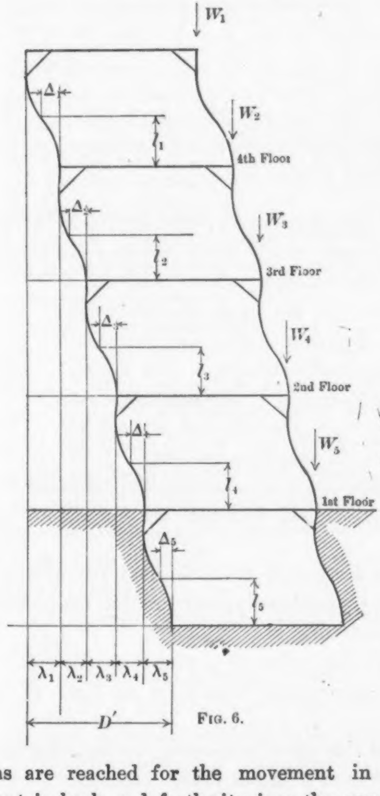


FIG. 6.

These conditions are reached for the movement in one direction. As this movement is back and forth, it gives the approximate curves shown by Fig. 7.

It can readily be seen that while the curve, $O\ A$, shows the curve in the building for the movement, $1 - A$, that, before the return movement throws the reverse curve, $O - B$, into the structure, a portion

of the frame at the top will endeavor to straighten, or that the point, O, will move to x, and that the curve on the beginning of the return movement will be $x\ y\ z\ A$. This tendency is aggravated on each reverse, and produces the whip action at the top.

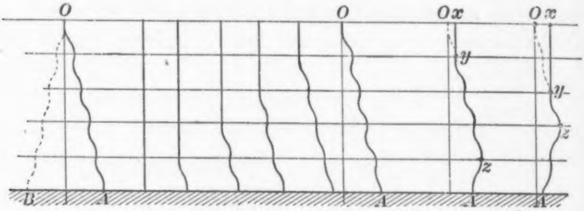


FIG. 7.

By Equation 5 it may be seen that should D be very small and l_s on the first story large, nearly all the bending would occur in the first-story columns, the building above receiving a very small force. In this case, the first-story columns vibrate back and forth, and the building above is practically stationary. Of course, this would be productive of a high bending movement in the first-story columns, and a shock of any magnitude would wreck the building.

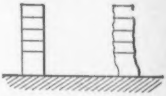


FIG. 8.

If the foregoing analysis is correct, the following may be noted:

1st.—That the stresses produced are similar to those caused by wind;

2d.—That, on account of quick reversal, the stresses are increased;

3d.—That, in a wind-braced structure, the total effect is distributed throughout the structure;

4th.—That, as this effect is a direct function of the weight, the wall and floor construction should be as light as consistent with strength;

5th.—That, as this effect is inversely as the coefficient of elasticity, the frame should be of a highly elastic material;

6th.—That, as the effect in buildings that are not wind-braced varies as the cube of the height, these structures should be limited in height;

7th.—That a monolithic foundation is preferable to one having isolated footings.

These conclusions all point to a steel frame with reinforced walls and floors as the type of construction for the vicinity of San Francisco.

With respect to reinforced concrete, the writer, although he believes it to be a valuable combination, thinks it unsuited to resist the forces that an earthquake shock would produce in a high building, for the following reasons:

1st.—This type of construction is not adapted to resisting reversed stresses;

2d.—It cannot take shock;

3d.—The construction is heavy, which conflicts with Conclusion 4;

4th.—The coefficient of elasticity is low, which does not agree with Conclusion 5;

5th.—The high bending moments produced in columns and girders would make their designs uneconomical;

6th.—Added to these, when it is considered that any failure in detail will necessitate the renewal of several entire members, the disadvantages of this construction will be seen.

Finally, the writer would recommend:

I.—The building to have lattice girders of the Warren type, as deep as the spandrel section will allow, running entirely around the structure at every floor. The advantage of this construction is obvious: Being in the center of the wall, the brickwork or concrete can be built around the members, the wall thereby being reinforced, the girder can be designed economically for the different floors. Being deep, it forms the lintel over the windows and at the same time decreases the length of the column.

II.—Monolithic foundations.

III.—Reinforced concrete walls, the reinforcement to run horizontally and vertically.

IV.—Floor slab to be of rock concrete at least $3\frac{1}{2}$ in. thick.

V.—Floors cut up by a large number of openings to be braced so that they can transmit all horizontal shear to the columns.

VI.—Wind bracing connections to be designed to develop the main member.

VII.—Wind bracing to be carried to the ground.

VIII.—Columns to be calculated for an extreme fiber stress of 12 000 lb.

DISCUSSION.

Mr. Waite.

GUY B. WAITE, M. AM. SOC. C. E. (by letter).—In the paper entitled "Wind Bracing for High Buildings,"* the writer assumed a horizontal wind pressure of 30 lb. per sq. ft. as acting against the entire windward side of the structure. The stresses induced by the wind force were assumed to be resisted by the construction acting as a cantilever. The building was assumed to be plumb. No increased bending moment, caused from an overhang, by wind pressure was thought necessary. Provided the weight of the building was sufficient to counteract the overturning moment due to wind, the mass or weight of the building did not enter into the discussion.

The resistance of the structure to the horizontal component of wind pressure was discussed without reference to whether it weighed 100 lb. or 1 000 000 lb. Mr. Chew's first conclusion, that the stresses produced by earthquakes are similar to those caused by wind, is misleading. While the force from wind pressure is definite, and is distributed throughout the structure, the force from an earthquake is indefinite and unmeasurable, and is distributed throughout the structure only by acceleration taken at one place—the foundation.

To compare the two forces, it may not be improper to liken them to the working of horizontal engines under a given pressure: the force of the wind being similar to small pistons with long strokes, while the force of the earthquake is similar to a piston having an indefinitely large area with a very small stroke. In the former case, the engine force is distributed against the side of the building, is limited in amount, and is substantially all taken up by the structure; in the latter case, the immeasurable force from the unlimited piston area is simply carried through the foundation of the building, the only force taken up by the building being due to the vibration of the foundation.

On the leeward side of a building the force of the wind is practically nil, while the force of the earthquake is substantially the same on each side of the foundation. The wind force is all taken by the building, whether it be a heavy or a light structure. The vibration of the foundation of a building, from earthquake (other things being equal), will be the same, whether it be heavy or light; the momentum only will vary with its weight.

The amount of the vibration of the superstructure of the building caused by earthquake will depend on the rapidity and length of stroke of the so-called piston, the elasticity of the material in the construction, and the design of the building. For instance, if the foundation of the building were divided into two horizontal parts, with roller or ball bearings between the parts, the earth vibration could pass through without materially affecting the upper structure, whether heavy or light, the only vibration being due to the friction of the bearings.

**Transactions*, Am. Soc. C. E., Vol. XXXIII, p. 190.

If the vibrations were sufficiently rapid, and the columns sufficiently long and elastic, probably little vibration would be felt, whether the building were light or heavy. Mr. Waite.

While some structural resistance is absolutely necessary in the case of wind—the force being definite and positive—the structural resistance required in the case of earthquakes will depend on circumstances and design, the force itself not being against the building.

Now, with a properly designed building, having a given mass and a certain given vibration of the foundation, but little vibration may be caused to the superstructure, while, with more unfavorable designs, vibration enough to wreck the building might be caused, even if the best of wind bracing were used.

Buildings may be definitely braced against wind pressure, but they cannot be definitely braced against earthquakes.

A pile of brick may be laid so that it will resist wind pressure, but will fall on account of the acceleration caused by an earthquake; this, however, is no reason why a sober fat man cannot stand up as well as a lean man under both earthquake and wind pressure. In Nature we see resistances increasing with the size and weight of objects; if this idea be carried out in buildings, we will but obey the mathematical laws which govern the stability of all things composed of matter. The heavier the building the more horizontal resistance it will naturally have; but the writer does not agree that the resistance should increase, as indicated in the rational analysis from which the author draws his conclusions.

It is generally considered that additional weight helps to distribute wind pressures, and that the force of the wind is largely used up in frictional and other internal work on the mass, to the relief of specifically designed resistances.

When a force is set up in a building, from the vibration of the mass composing the foundation, why should not much of this force, which would otherwise be communicated to the braces and connections of the superstructure, be lost in doing the internal work referred to above?

As buildings can be properly designed to resist wind, and can only be designed to escape the vibration of earthquakes, it is believed that they should be constructed so that their parts will withstand wind pressures, and that they will then be amply provided to withstand earthquakes.

Wind pressures are very frequent, while earthquakes are very rare. A building will have need for resistance to wind pressures several thousand times for one possible resistance to earthquake.

There is no evidence to show that a modern wind-braced building is not strong enough to withstand the vibration from an earthquake, but there is considerable evidence to show that it is sufficient.

Mr. Waite. A building well designed to resist the force of the wind should have plenty of good-sized columns. The connections and braces of these columns to the girders should be strong and positive, and there should be the maximum depth of connections of cross-beams and girders to columns. All constructions composing floors, walls, partitions, etc., should be capable of distributing stresses and of withstanding vibrations.

It seems to the writer that reinforced concrete fulfills these conditions better than any other known material. In reinforced concrete the columns and girders have a monolithic connection throughout their height, and, with proper design, can take stress in both directions. The concrete, being a filling between the reinforcing steel, can take any amount of vibration which will be conveyed by an earthquake. The reinforcing steel is run into the columns, making a stronger connection than possible in steel and ordinary fire-proof construction.

If there be any part of a building in which concrete properly reinforced cannot be designed as light as, and perform the function of resisting stresses better than, any other fire-proof material (in addition to which it preserves the steel), the writer is not aware of it.

Mr. Walker. E. G. Walker, Jun. Am. Soc. C. E. (by letter).—The writer has read with very great interest Mr. Chew's paper and the analysis with which he endeavors to arrive at the facts of the resistance of a steel-framed building to earthquake shocks. At first sight it would appear that this subject is not one which is susceptible of much calculation, but, when the nature of an earthquake disturbance is considered more closely, it at once becomes apparent that a rational analysis may easily be made.

As Mr. Chew remarks, the effect of an earthquake is a wave motion. The motion usually commences with small elastic earth-vibrations of short periodicity, followed later by the shock proper, the period of which will be much greater, from 1 to 2 sec., after which the vibrations will become slower and smaller until a quiescent state is again reached.

This being the nature of the force acting on any structure during an earthquake disturbance, the writer does not think that the author's analysis, though correct and in order as far as it goes, is sufficiently extended to take true cognizance of the initial conditions of the problem. Mr. Chew treats his building as an elastic structure, the foundations of which are subjected to an impact by a seismological wave, but he neglects the fact that this wave is followed by others at intervals which, for a short period, may be regarded as regular.

When an elastic body is struck a blow, it tends to vibrate with its own natural period of vibration, and the amplitude gets less and less until the body comes to rest, the maximum distortion, and therefore, also, the maximum stresses, occurring just after the impact. This is the state of affairs Mr. Chew assumes.

Mr. Walker.

The writer submits, however, that to treat the problem on this basis is insufficient. The building should be considered, not as merely subjected to an impact, but as acted upon by a force the intensity of which varies according to a regular periodic law. The building is then compelled to execute forced vibrations, and, if it should happen to have a natural period which synchronises with that of the applied force, a resultant vibration of very large amplitude would be set up, causing extreme stresses. On the other hand, and this, presumably is a more common case, if the natural period of the building and that of the impressed force are different, there would be a continuous variation of stress in all members of the structure though the range would not be so great.

The mathematical treatment of the problem, on these lines, though a little complicated, only follows the orthodox method of investigations into the ordinary problems relating to vibrations. The writer had intended to present a solution in some of the simpler cases, such as those dealt with in the paper, but, unfortunately, the time at his disposal has been insufficient to take up the question in detail. However, it should be possible, by treating the structure as a vertical cantilever acted on by a periodic force, to arrive at a law of displacement for any portion, and thus to get a value for the maximum deflection, $\angle$. at any point, as well as an accurate knowledge of the range, or extreme values of deflection. The step from this to a calculation of the stresses induced is easy and straightforward.

With regard to the author's first conclusion, it seems to the writer that the stresses produced by a shock will be similar to those caused by wind only in cases where the wind pressure is produced by gusts at fairly regular intervals. The increase or otherwise of stresses mentioned in his second conclusion would be brought out in an analysis on the lines the writer has mentioned, and, in calculating the scantlings of a new structure, the extreme fiber stress allowed could be settled in accordance with the range of stresses found.

This subject of the stresses induced in an elastic structure by seismological disturbances is a very interesting and important one, from both practical and theoretical standpoints. Up to the present, rules and formulas have been mainly the outcome of observation and experiment, rather than of deduction from the theoretical investigation; but there is no reason why the latter method should not be used; so that, with a knowledge of the seismograms which have been recorded from time to time, it should be possible to deduce, with a fair degree of accuracy, the probable stresses which would be induced by a shock, and to provide suitably for them. The writer only regrets that he has been unable to devote the time necessary to work out an analysis on the lines he has endeavored to indicate.

Mr. Stern. EUGENE W. STERN, M. AM. SOC. C. E.—Earthquake shocks vary so much in intensity and character that it seems improbable that any assistance can be gained from mathematical investigations, in the practical designing of buildings to withstand these shocks.

Along a fault line, and even many miles away, the force may be irresistible, owing to conditions which cannot be controlled, such for instance as deformation of the soil; and the effects on buildings may be such that damage is inevitable. How to minimize this, so as to cause the least loss of life and property, is the vital problem.

The effects which earthquake shock produces on the earth may be divided into:

1.—Elastic vibration,
2.—Change in contour.

If a building were designed to withstand only elastic vibration, using a system of bracing similar to wind bracing, rigidly connected to columns and girders, as recommended by the author, the damage to buildings would be very serious if the change in contour of the underlying soil also occurred. For this reason the speaker cannot agree with the author's recommendation of this type of bracing.

The speaker would recommend a structural frame of steel, in which the column splices are so thoroughly well designed as to make the columns practically continuous from bottom to top; a system of curved knee-braces made of two angles, connecting girders to columns, strong enough to resist wind pressure, but not stiff enough to deform columns or girders in case of unequal settlement of foundations; and flexible connections between girders and columns, which would allow such movements to take place. The curved knee-braces, in such an event, would bend without injuring the columns or girders, and the building could be restored to its proper level by using wedges and hydraulic jacks under the columns.

This principle of construction has been used by the speaker in the structural design of the Hoboken Terminal of the Lackawanna Railroad. The conditions which had to be met in this structure were that it should be fire-proof, that it should be designed so that unequal settlement of the foundations would not injure the steel frame or the walls, and that it should be capable of withstanding the vibratory shock due to the accidental ramming of a heavy ferry-boat. The conditions, therefore, approximated on a small scale those existing during an earthquake, and the system of curved knee-braces mentioned above was used. The walls and floors were made of reinforced concrete securely tied to the steel frame. This method of construction has proven entirely satisfactory in every way. Settlements of as much as 5 in. have occurred in the foundations without injuring the steel frame or walls of the structure. The speaker was led to develop this

method of construction by conclusions derived from an examination of old pier sheds supported on poor foundations, along the river fronts in New York. Some of these had stiff riveted connections between girders, trusses, and columns. One case was of significant interest, the columns being bent almost 2 in., due to the twisting action of the girders connected thereto, on account of unequal settlement of the foundations under the columns. Mr. Stern.

The recommendations of the author, Nos. III and IV, regarding reinforced concrete walls and floor slabs, coincide with the speaker's views and recommendations in his discussion on "The Effects of the San Francisco Earthquake."* The speaker believes in the use of steel girders and joists in the floor system to avoid long-span reinforced concrete slabs.

The author makes no recommendations as to a very vital question, namely, limitation of the height of buildings. It is not necessary to state that the knowledge in the hands of engineers to-day permits them to erect structures of almost any height, but a very important question arises: whether it is proper, from the standpoint of those inhabiting them, to construct high buildings in districts subject to earthquake visitations. The speaker submits that it is indisputable that the lower the building the safer it can be made, and the more easily repaired in case of damage. What the limiting height should be is of course a question for consideration. Real estate interests would oppose such a limitation, but the more important consideration, that of the safety of the public, should be paramount.

In high buildings the difficulty of providing proper methods of construction to withstand earthquake shocks, of rectifying damages to foundations, etc., and of controlling the conflagrations which usually accompany these disasters, are matters of such vast importance to the welfare of the community that real estate interests should be made subservient to them.

R. S. Chew, Assoc. M. Am. Soc. C. E. (by letter).—In reference to Mr. Waite's illustration as to the working of a horizontal engine, the writer cannot see where the results are not similar. Although it is true that there is no wind on the leeward side of a building, yet there is a sheering stress transferred through by the girders, so that both windward and leeward columns on a narrow building are assumed to get wind. Wind attacks the structure throughout its height (the forces acting at the several stories) and produces a horizontal reaction at the base. Mr. Chew.

Earthquake shocks reverse this, attacking the building at the base, producting this reaction, as it were, and, by deformation, forcing stresses throughout the building to counteract it. The effect on the structure is the same in both cases. Of course, as stated in the paper,

* *Transactions*, Am. Soc. C. E., Vol. LIX, pp. 273 *et seq.*

Mr. Chew. the effect is aggravated by the return movement of an earthquake. With respect to vibration, Mr. Waite's idea coincides with the theory in the paper, but the writer fails to see how a properly braced building can escape. With a given movement of the foundation there appear to the writer to be only three results: First, if the columns in the first tier are long and not kneed, they would take nearly all the movement, and probably fail, as noted in the paper; second, if the building is properly braced, the total movement is distributed through the structure; third, if the building were monolithic, it would attempt to follow the movement as a whole, and thereby fail by the heavy shearing forces produced. The illustration of a pile of bricks would come under the third class.

Again, in wind bracing, it is assumed by some designers that 50% of the wind stress is resisted by the floor, or internal work. This simply means that certain portions of the structure are not figured, but do resist. In an earthquake, every column gets its portion of work to do in proportion to the weight carried, so that each brace and connection is called upon to perform its function; therefore, there is little chance for any force to be lost in internal work.

With respect to wind pressures being frequent and earthquakes rare, it may be noted that the writer advocates a structure thoroughly braced for wind.

As to reinforced concrete buildings, the writer agrees with Mr. Waite that they can be made with monolithic connections, but is still of the opinion that they are unsuited for earthquake strains.

In reference to Mr. Walker's remarks, the writer does not believe that any deeper analysis will aid.

Mr. Stern's recommendation for a knee brace does not appeal to the writer. Although it has undoubtedly fulfilled its function in the design spoken of, yet the writer does not see how it would act in a high building. If a building happens to be on a fault line, it would be wrecked by a shock. The writer's assumption was that the foundation settled equally, if at all; he purposely omitted any reference to the height. The Call Building frame (eighteen floors) withstood both earthquake and fire, and is an example of what a steel frame can do.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1083

ERECTION OF THE BELLOWS FALLS ARCH BRIDGE.*

By Lewis D. Rights, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. J. R. Worcester, J. P. Snow, F. W. Skinner, Henry H. Quimby, Philip Aylett, and Lewis D. Rights.

The highway bridge across the Connecticut River at Bellows Falls, Vt., is interesting because, in the United States, it stands alone as an example of a through arch with suspended floor, and also because, as an arch, it is only surpassed in span by the two deck arches at Niagara.

The residents of North Walpole, N. H., depend largely on the factories at Bellows Falls, Vt., for their employment, and on the stores for their trading. To reach the town, they were compelled to use the old wooden toll bridge at the south end, or venture on the Sullivan County (Boston and Maine) Railroad bridge, or patronize a rather uncertain rowboat ferry. For years they had urged a more convenient crossing, and this was naturally backed up by the merchants and business men on the Vermont side. The depth of the river at this point, about 25 ft., strong objections to piers above the mouth of the canal, owing to the vested rights of the Canal Company, and the freeing of the old toll bridge before another could become available, were factors contributing to the delay of the project, which resolved itself, largely, into a matter of cost.

* Presented at the meeting of April 1st, 1908.

Early in the spring of 1904, the agitation was again revived, and interested citizens brought forward new and old schemes. One that met with considerable favor was to locate a pier on the rock in shallow water just above the angle of the dam, shown on the map, Fig. 1. This permitted the use of two spans, but made it necessary that these spans should be at an angle with each other in order to reach convenient landing places on the shores.

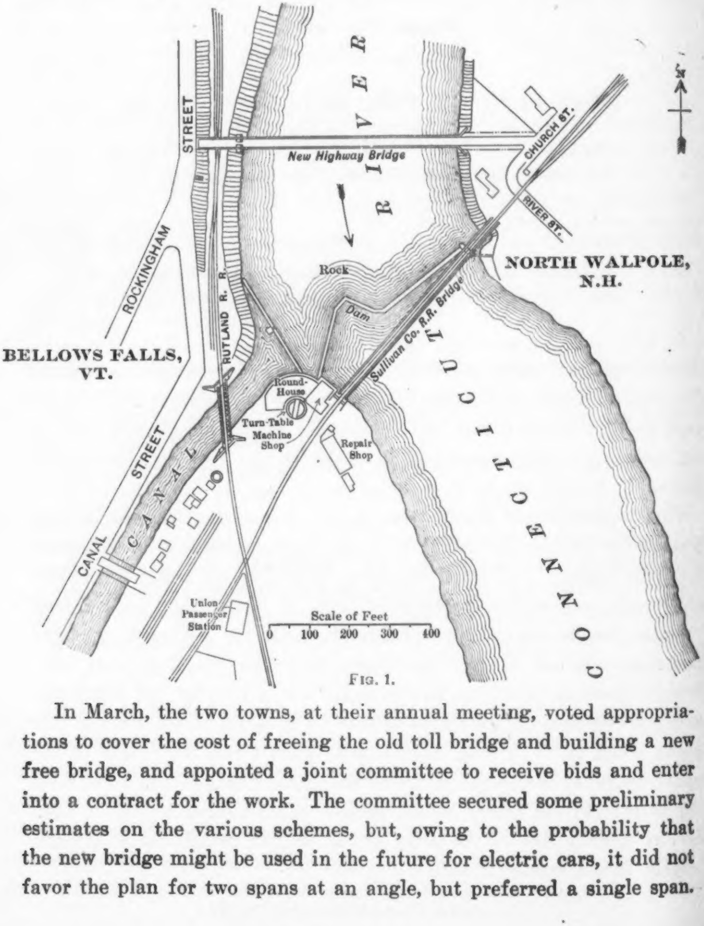
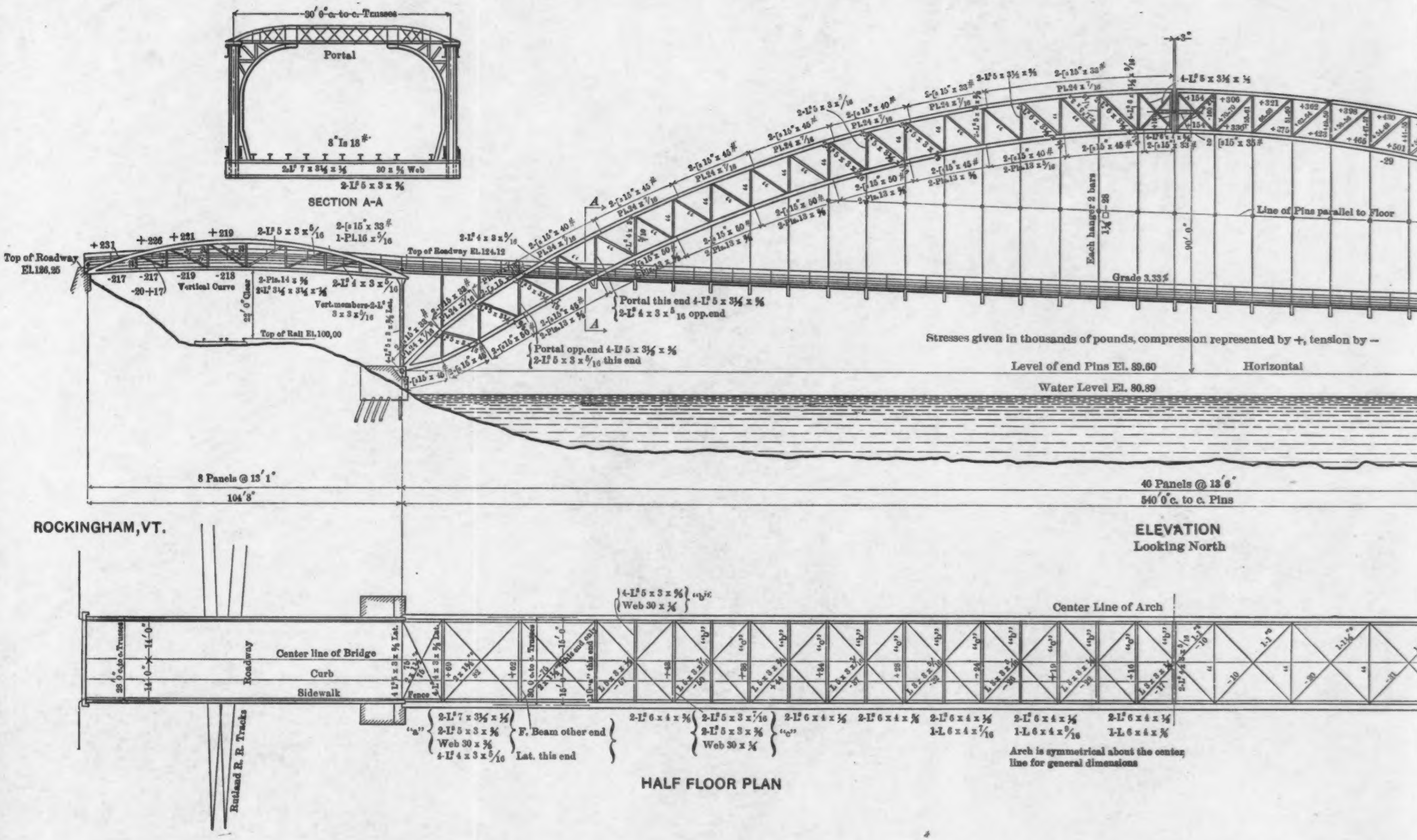
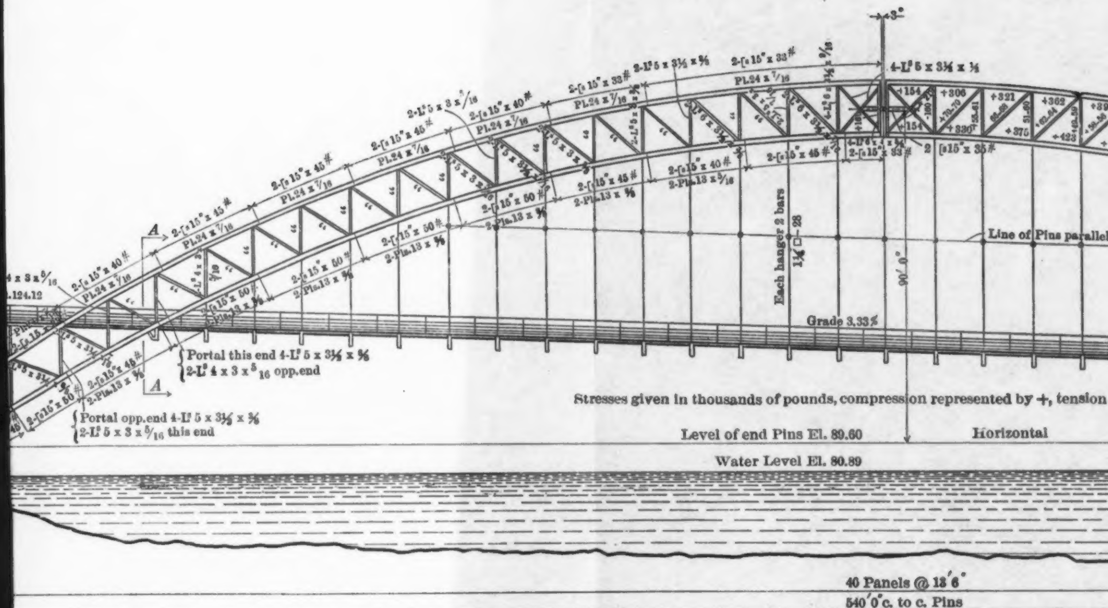


FIG. 1.

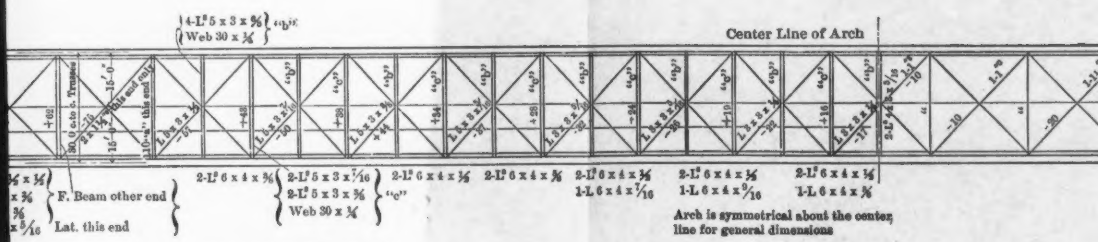
In March, the two towns, at their annual meeting, voted appropriations to cover the cost of freeing the old toll bridge and building a new free bridge, and appointed a joint committee to receive bids and enter into a contract for the work. The committee secured some preliminary estimates on the various schemes, but, owing to the probability that the new bridge might be used in the future for electric cars, it did not favor the plan for two spans at an angle, but preferred a single span.



SECTION A-A
Portal
30' 0" c. to c. Trusses
ROCKINGHAM, VT.
8 Panels @ 13' 1"
104' 8"
40 Panels @ 13' 6"
540' 0" c. to c. Pins
ELEVATION
Looking North
Top of Roadway El. 126.25
Top of Roadway El. 124.12
Top of Rail El. 100.00
Vertical Curve
Line of Pins parallel to Floor
Grade 3.33%
Stresses given in thousands of pounds, compression represented by +, tension by −
Level of end Pins El. 89.60
Horizontal
Water Level El. 80.89
Portal this end 4-L^s 5 x 3½ x ⅜
Portal opp. end 4-L^s 5 x 3½ x ⅜
Each hanger 2 bars
Center Line of Arch
Center line of Bridge
Curb
Sidewalk
Roadway
Fence
Rutland R. R. Tracks
F. Beam other end
Lat. this end
Arch is symmetrical about the center line for general dimensions
HALF FLOOR PLAN

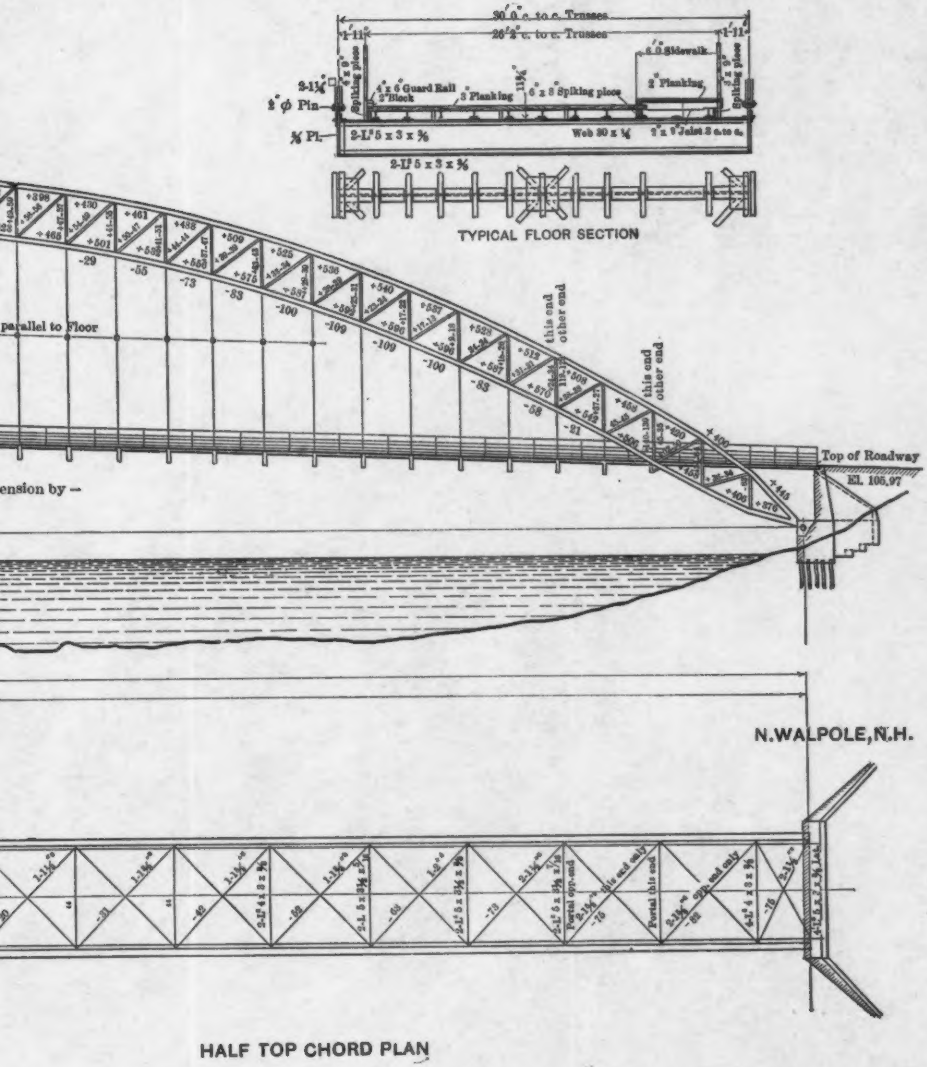


ELEVATION
Looking North



HALF FLOOR PLAN

PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1083.
RIGHTS ON
ERECTION OF ARCH BRIDGE.



As the preliminary estimates for a single-span bridge were unsatisfactory, some members of the committee visited Boston and appealed to the President of the Boston and Maine Railroad, who was interested to the extent of freeing the railroad bridge from unauthorized foot passengers, and who responded by offering the services of J. P. Snow, M. Am. Soc. C. E., in an advisory capacity. Mr. Snow was naturally familiar with the general surroundings, but did not feel that conditions warranted the preparation of an elaborate design; therefore, he drew up general specifications, and called for bids, requesting each bidder to submit his own plans. Among the designs submitted were several for truss and suspension bridges, but all the prices were greater than the appropriation, and the bids were rejected. Mr. Snow was satisfied that a bridge could be built within the specified sum, and recommended the employment of J. R. Worcester, M. Am. Soc. C. E., which suggestion the committee accepted. Mr. Worcester concluded to adopt an entirely different type of bridge, and decided that a three-hinged, riveted arch with suspended floor would be the most artistic and suitable structure for the location. He drew plans and specifications, and, on his recommendation, separate bids were asked for the masonry and structural steel. The results of the competition were satisfactory, as several bids were received which were within the appropriation. On the recommendation of Mr. Snow, the contract for the masonry was awarded to Joseph Ross and Sons, of Boston, and the superstructure to Lewis F. Shoemaker and Company, of Philadelphia and New York.

Design.—As will be seen by the general plan and stress sheet, Plate XXXIII, the bridge is about 650 ft. long, and consists of a single, three-hinged, arch span, 540 ft. from center to center of end pins, with a short truss span at the west end, 104 ft. 8 in. from center to center of bearings. This short span was necessary, in order to carry the street over the Rutland Railroad. It will be noted that the roadway is on a grade of 3.33%, running downward from the short span to the abutment at the east end. The height of the main arch is 90 ft. between the hinge centers. The truss chords follow the lines of two parabolas 14 ft. apart. In order to secure simplicity of detail, the trusses do not diverge at the bottom, but stand in parallel vertical planes, 30 ft. from center to center. This provides for a roadway, 20 ft. clear, and one sidewalk, 6 ft. wide, as shown by the cross-section.

At the east end of the arch an abutment has been provided, and at the west end two piers take the thrust of the main arches, and support the vertical posts which carry the end of a small truss bridge.

The floor is designed for either a live load of 100 lb. per sq. ft. or a 12-ton wagon load on two axles, 10 ft. from center to center. In addition, provision is made in the hangers and floor beams for a single-track line of 18-ton electric cars, to run on the opposite side of the bridge from the walk. The connections for track stringers have been provided in the floor beams, and, if it should be thought advisable in the future to carry the trolley line over to the New Hampshire side, it can be done at a very small expense.

The main trusses are designed for a live load of 60 lb. and for a wind load of 40 lb. per sq. ft. The floor is suspended from the arches by two hanger bars, $1\frac{1}{4}$ in. square, at each panel point. These are connected, with 2-in. pins, to the truss at the top and the floor beams at the bottom. The hangers can only take the vertical load, and therefore wind chords have been provided in the planes of the trusses at the level of the roadway. These chords, with the floor laterals, form a horizontal truss which carries the wind stresses to the abutments. To prevent complication in the arch stresses, this lateral system is not attached to the arches rigidly, but has expansion joints at each end transmitting shear only.

The unit stresses adopted were:

For tension:

Steel for floor	15 000
Steel for trusses	16 000
Laterals	17 000
Wrought-iron loop rods	9 000

For compression:

Main arch chords	15 000
Struts	$\dfrac{13\,500}{1+\dfrac{l^2}{36\,000\,r^2}}$

For pins and rivets:

Shear	10 000
Bearing	18 000
Bending on extreme fiber of pins	22 000

Allowable bearing stress for concrete, 400 lb. per sq. in.

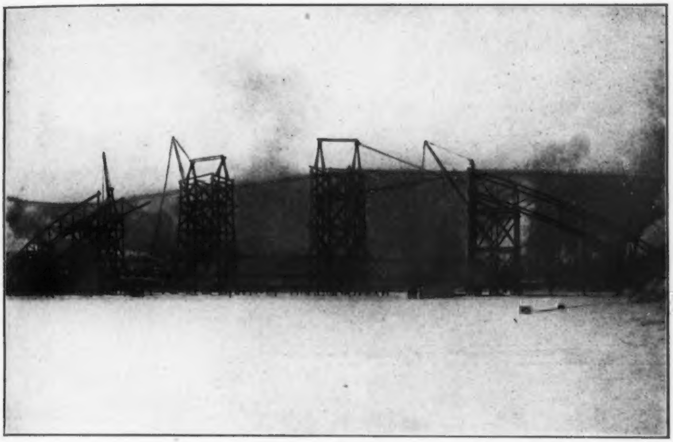


FIG. 1.—BELLOWS FALLS ARCH BRIDGE IN PROGRESS.

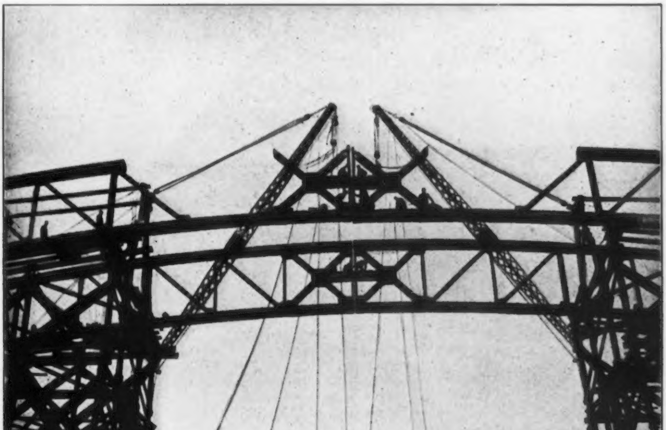


FIG. 2.—CONNECTING ARCH AT THE CENTER.
(CAMERA POINTED UPWARD AT AN ANGLE OF 45 DEGREES.)

The steel was furnished in accordance with the specifications of the American Railway Engineering and Maintenance of Way Association, the ultimate tensile strength being 60 000 lb. per sq. in. The flooring timber is long-leaf Georgia yellow pine of prime quality.

One of the unique features of the design is the arrangement which Mr. Worcester has adopted for the middle of the arch. It will be noticed that, instead of converging the chords at the center panels, in order to take the stress of the center hinge, he has taken the horizontal thrust on a short strut, composed of two 15-in. channels, and has then divided this thrust into four parts at the intersection of the diagonals in the panels next to the center. In this way he has secured the architectural features of a two-hinged arch with continuous chords, adding considerably to the appearance of the bridge. It will also be noticed that he has dispensed with the pin at the center, the struts merely bearing against one another with faced ends.

In order to avoid the optical illusion of a sag in the suspended floor, this was cambered 18 in. at the center of the arch. It was felt that this camber would take care of any slight deflection due to live load, and the design and details of the arch were developed on theoretical lines, with no provision in the arch for either live- or dead-load camber.

Erection Design.—In securing data for the preliminary design for the estimate of the cost of erection, the idea of building towers some distance apart and erecting the trusses by the cantilever method, suggested itself to the writer, the main question being to determine the economic spacing of these towers. As it was not considered advisable to do any riveting until the arch was swung, it was necessary to plan to bolt all connections temporarily. The strength of these bolted connections, therefore, was a factor in the length of the cantilever. Another consideration was the distance that material could be handled satisfactorily with a standard 60-ft. steel boom. The writer made a design, and decided that a span of six panels, or about 80 ft., would give the best results. The sizes and details were then developed in the drawing room, under the direction of the chief engineer.

On account of the narrow width of the structure in comparison with its height, the center falsework towers were battered in the plane at right angles to the bridge, thus enabling them to withstand better the shock of winter storms or floating ice. The general elevation of the falsework towers is shown on Plate XXXV, which indicates the

number and spacing of the pile supports. The details of the center bents, F, F_1, and F_2, are shown by Fig. 2, which gives the size of the main posts and bracing, and indicates the splices and number of bolts.

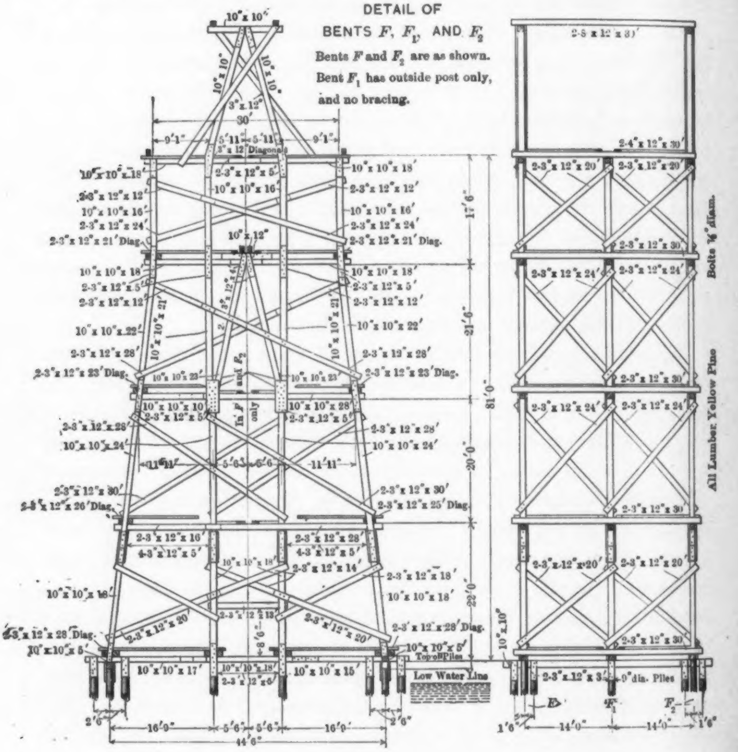
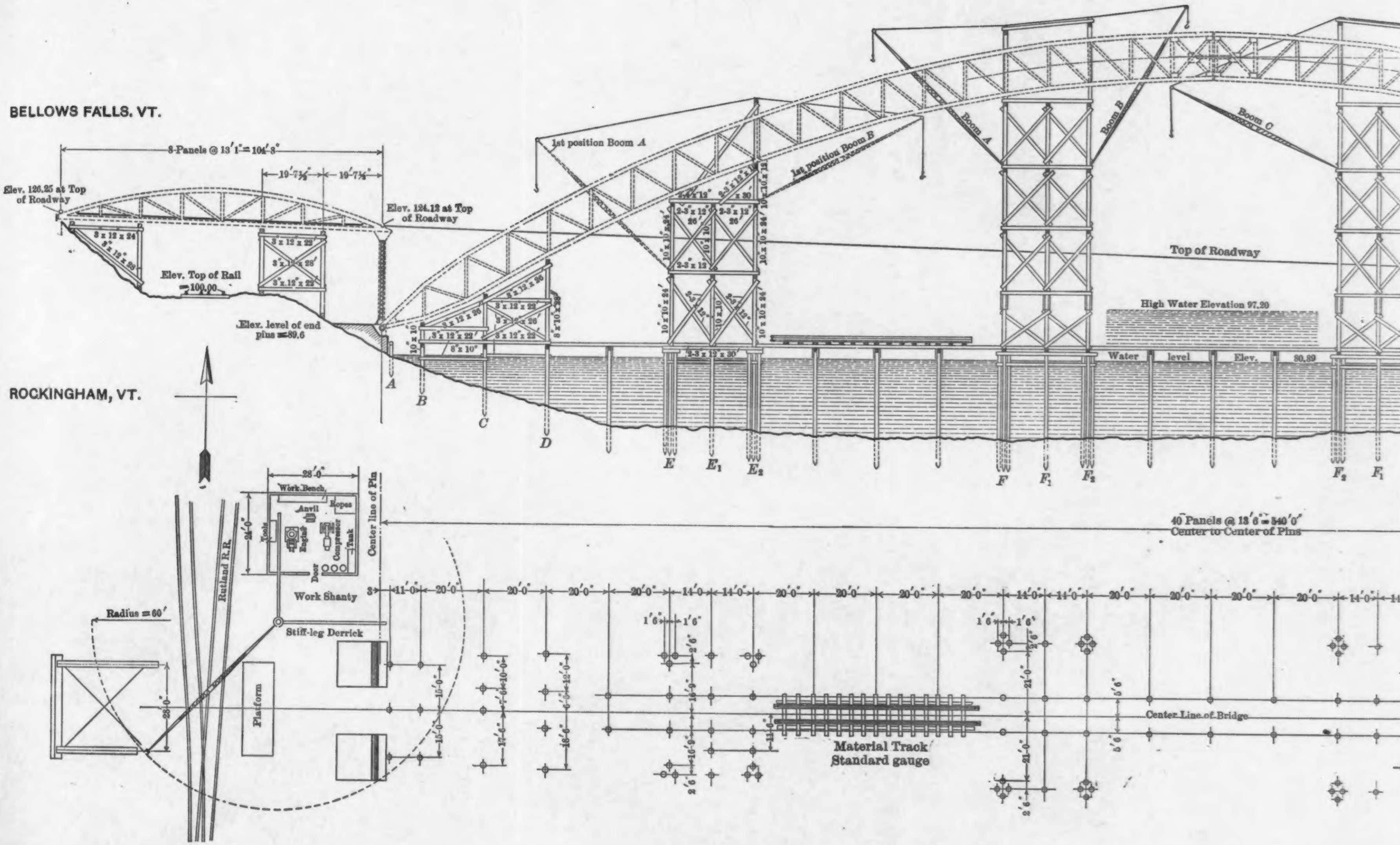
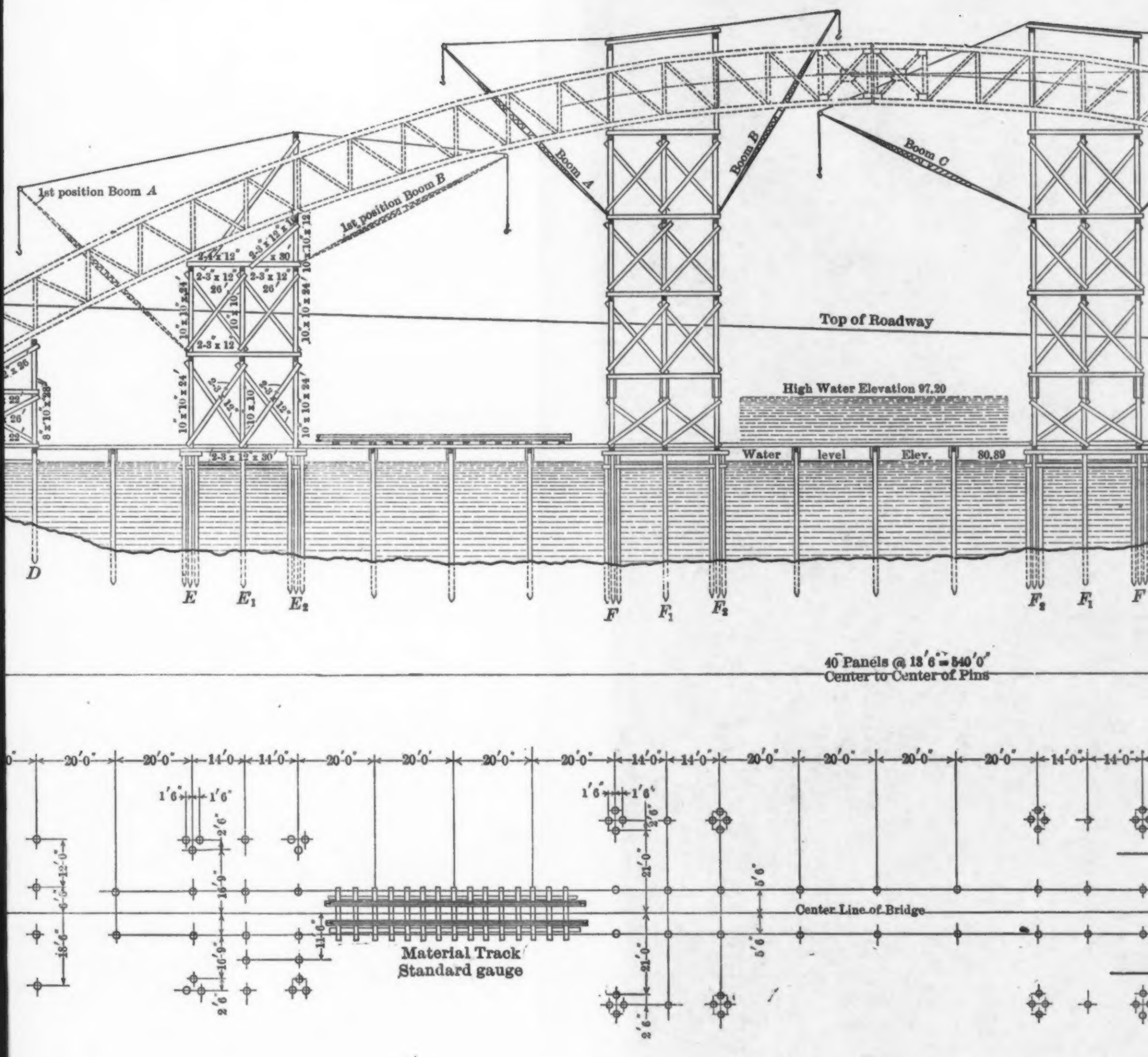


FIG. 2.

It was considered advisable, on account of the switching facilities, to unload the material on the Vermont side of the bridge and carry those pieces required for the east side on a standard-gauge service track running between the towers, along the center line of the bridge. A 10-ton stiff-leg derrick was placed on the west shore, to unload material from the cars and transfer it to the trucks on the service track. Two 30-h.p. hoisting engines, with two drums and four spools each, were located at Bent E to raise the steel.

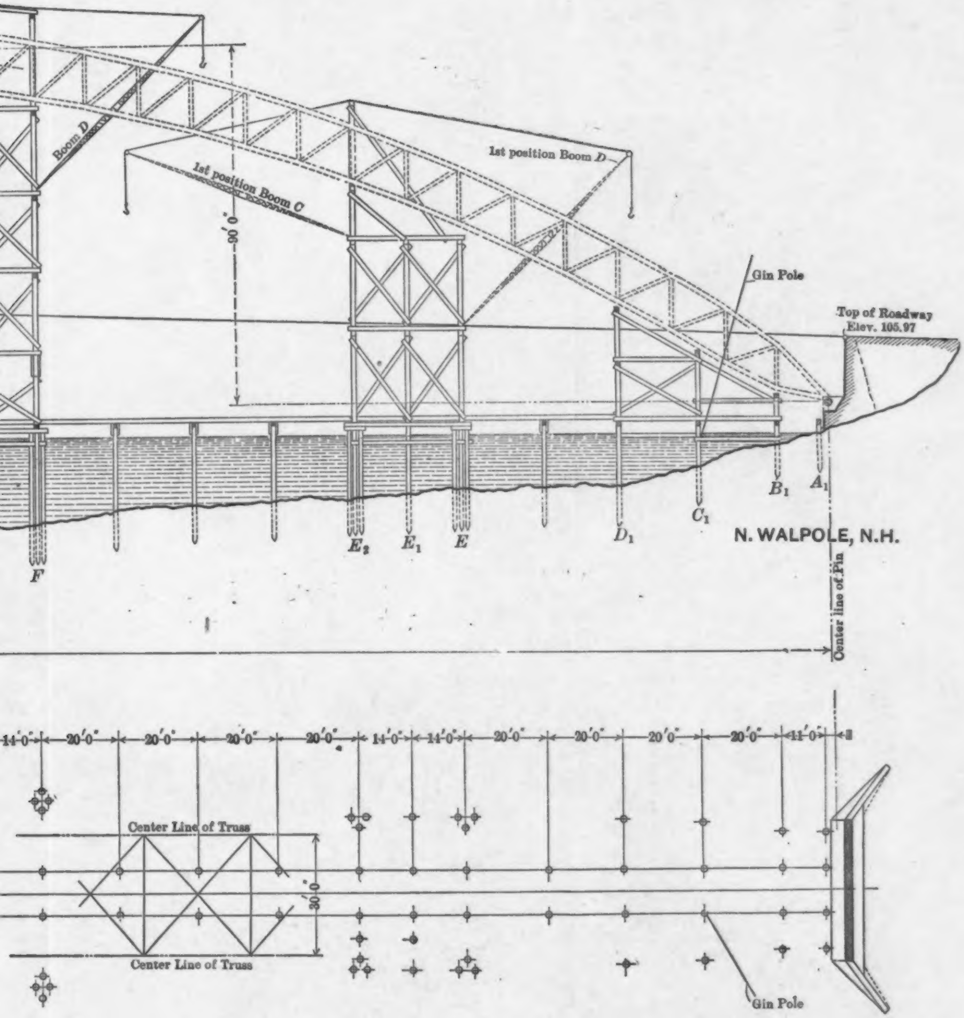


BELLOWS FALLS, VT.
ROCKINGHAM, VT.
8-Panels @ 13′1″= 104′-8″
19′-7½″
19′-7½″
Elev. 126.25 at Top of Roadway
Elev. 124.12 at Top of Roadway
Elev. Top of Rail =100.00
Elev. level of end pins =89.6
1st position Boom A
1st position Boom B
Boom A
Boom B
Boom C
Top of Roadway
High Water Elevation 97.20
Water level Elev. 80.89
A
B
C
D
E
E1
E2
F
F1
F2
40 Panels @ 13′6″= 540′0″ Center to Center of Pins
Center line of Pin
28′0″
24′0″
Work Bench
Anvil
Ropes
Tools
Engine
Compressor
Tank
Door
Work Shanty
Rutland R.R.
Stiff-leg Derrick
Radius = 60′
Platform
Material Track Standard gauge
Center Line of Bridge



1st position Boom A
1st position Boom B
Boom A
Boom B
Boom C
Top of Roadway
High Water Elevation 97.20
Water level Elev. 80.89
D
E
E1
E2
F
F1
F2
40 Panels @ 13'6" = 540'0"
Center to Center of Pins
Center Line of Bridge
Material Track
Standard gauge

PLATE XXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1083.
RIGHTS ON
ERECTION OF ARCH BRIDGE.



N. WALPOLE, N.H.

Erection.—The masonry plans provided for piles to be driven in the foundations for the arch, and, as the masonry contractor was well equipped to do the work, the contract for furnishing and driving the piles for the falsework was sublet to him. The water has an average depth of about 25 ft., and the bottom is hard gravel. Spruce piles were specified, in order that they might be sold to the pulp mills after the work was completed. They were driven from 8 to 10 ft. into the bottom, and were cut off and capped about 3 ft. above the low-water line. The falsework towers were completed in November, and the shoes were set on December 6th, 1904.

Two gangs were started, one from each end of the arch, and the rivalry between them helped not a little in the rapid erection of the work. The severe weather of the winter of 1904-05 will no doubt be remembered, but, even in that latitude, there were breaks in the cold, and on two separate days considerable rain fell. The fear that the ice might go out, which would mean taking out the falsework and everything with it, was a constant incentive to hasten the erection in every way possible. It had not been the intention to do any work on the ice, but after it had frozen to the thickness of about 2 ft., it was found to be very convenient when assembling the chords, which were handled in two sections of four panels each. The ice also acted as a hindrance, for when the canal gates were closed, on Sundays and holidays, the river rose about 18 in., and it was necessary to keep the ice chopped free from the piles.

When the ice first began to form about the falsework, the structure showed a tendency to move down stream. This was carefully noted, account of it being taken in placing the steel. When the arch was swung, some of the bents were found to have moved down stream about 4 in. As the load was put on the falsework, some of the bents sank slightly, but this settlement was adjusted with wedges under the blocking at each point of support.

The two end panels of the lower chord and the end panel of the upper chord were shipped riveted together. These members, each weighing about 8 tons, constituted the heaviest pieces to be handled.

In beginning the erection, the shoes and end panels of the west end were set with the stiff-leg derrick used for handling material from the siding to the material trucks. On the east end a gin pole was placed to set the shoes and end panels. Provision had been made in the plans

for a clearance of 4 in. behind the steel shoes. As soon as the shoes and the first panels had been set and lined up, this space was filled with concrete.

Two standard steel booms, *A* and *D*, Plate XXXV, of 10 tons capacity, were erected at Bents *E*, and with these the steelwork was erected so as to rest on top of the falsework at these bents. Booms *B* and *C* were then erected at Bents E_2, and, with the assistance of the first two booms, the structure was laid across the *E* towers. Booms *A* and *D* were then taken down and re-erected at Bents *F*, thus giving two booms to handle the material over the 80-ft. space between Bents E_2 and *F*. The arrangement of the booms in this last position is shown clearly on the progress photograph, Fig. 1, Plate XXXIV. The lower chords for two sections of four panels were assembled on the ice, with the first diagonal at Bent E_2 attached to them. Then, while Booms *A* and *D* held up the material, Booms *B* and *C* were used to fill in the web members and put on the top chord. The last section of bottom chord from the cantilevered end to Bent *F* was placed by Booms *A* and *D*, which filled in the web members and the top chord. While this was being done, Booms *B* and *C* were taken down from their first position and re-erected at Bents F_2. The steelwork was carried across the *F* towers, and the erection of the 80-ft. space between them was taken care of by these two booms. The photograph, Fig. 2, Plate XXXIV, was taken with the camera pointed almost directly up at the work, and shows the connection of the lower chords of the arch members at the center.

To overcome the natural tendency of the chords to lengthen during erection, and to adjust any slight errors in the measurement between the end-pin centers, the details at the center were arranged to allow a slight leeway at the meeting points. The chords and the vertical posts at the center were set back 2 in. on each half from the center line, making a total clearance between them of 4 in. The center strut projected a little beyond the vertical posts, but was set back 1½ in. from the center line on each half, making a total clearance of 3 in. between the bearing points. With the intention of being able to take up the theoretical opening of 3 in. and for any adjustment that it might be necessary to make, 6 in. of fillers were provided. When the trusses reached the *F* tower, it was found that the erector had failed to make sufficient provision for the settling of the falsework at Bents *D* and *E*.

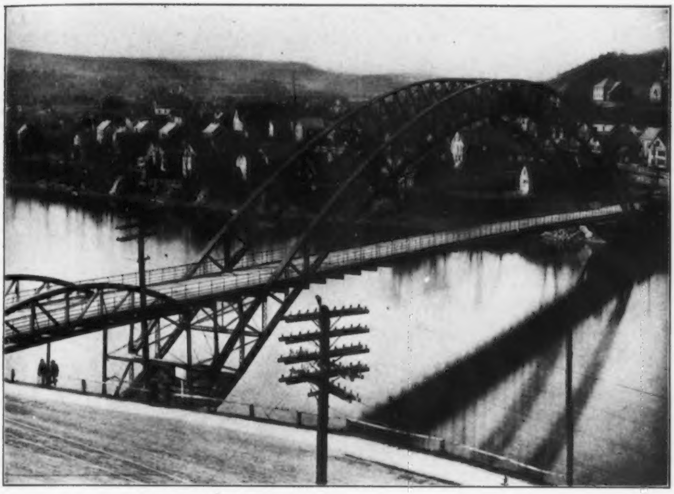


FIG. 1.—THE BELLOWS FALLS ARCH BRIDGE.

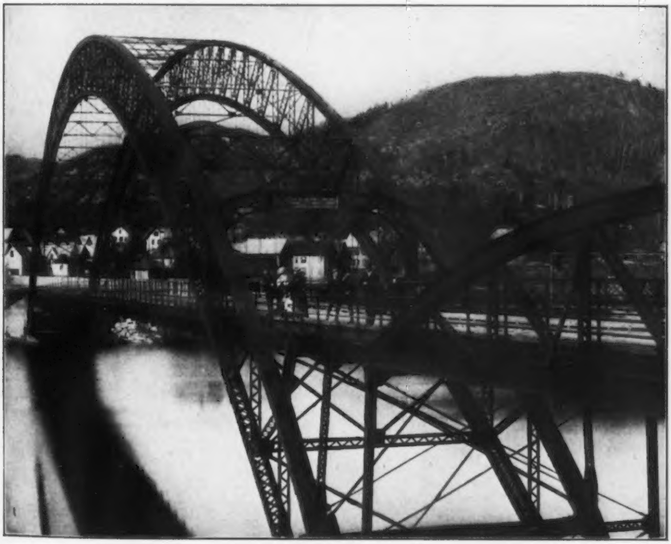


FIG. 2.—END VIEW, BELLOWS FALLS ARCH BRIDGE.

The result was that the arch was low, and measurements showed that the steelwork would overlap at the center. The trusses were raised to their proper height by wedges at the tops of all the towers. When the steelwork was joined at the center, it was found that it had slightly over-run the calculated distance. Instead of 3 in. of fillers, theoretically needed, a thickness of only 1½ in. was required.

As soon as the work was connected up it was thoroughly bolted. The web members were fastened with bolts in 75% of the holes, and the chords with bolts in 50% of the holes. When this was done, the wedges were knocked out and the arch ribs were swung off. As the only load the arch ribs had to support was that of the steel in the trusses, the deflection at the time of swinging off was comparatively small. The erection superintendent reported that the settlement at the crown was about 1½ in.

The trusses were connected and swung off on January 10th, 1905, making a total of 35 days from the time of setting the shoes until the arches were swung off. From this time should be deducted one holiday, three Sundays, and three days of cold or wet weather, which gave an actual working period of 28 days. During a good portion of this time the thermometer was about at zero, which made it difficult for the men to work actively.

The falsework was taken down the day after the arches were swung off, and the erection gang started from both sides to put in the hanger rods and place the steelwork of the floor system. The wood floor was not put on at this time, so that no local deformation of the arch ribs was noticed. While the steel floor system was being placed, other gangs adjusted the bracing and lined up the arches. It had been the intention to do the field riveting with pneumatic riveters, but a day before the riveting was to begin, the tool-house was destroyed by fire, and the compressor was injured so that it could not be used for this work. Eight gangs, of four men each, drove the field rivets by hand. As soon as the weather would allow, the field painting was completed and the wood floor laid. The finished bridge is shown by Figs. 1 and 2, Plate XXXVI.

Thirty-six men were employed when the erection of the steelwork began, and the number was increased afterward to forty-five. The weight of the structural steel was 450 tons for the main arch span, and 75 000 ft., B. M., of lumber were used in the floor. The contract price for the masonry was $6 000, and for the superstructure $41 000.

Credit for the work should be given to Mr. J. H. Fichthorn, Chief Engineer for Lewis F. Shoemaker and Company, and to Mr. A. L. Westbrook, Field Superintendent. The writer also wishes to express his thanks to Messrs. J. P. Snow and J. R. Worcester for the information furnished for the preparation of this paper.

DISCUSSION.

J. R. Worcester, M. Am. Soc. C. E. (by letter).—In connection Mr. Worcester. with the author's description of the ingenious method of erection of the structure, devised by himself, he has covered quite fully the design of the bridge and the conditions which led to the adoption of this type. He has generously omitted reference to one bad feature of the design, however, which should be recorded.

It may possibly be surprising to some that a design for an arch should have proved more economical than for a suspension bridge. The reason for this, however, is apparent, upon consideration of the profile through the location. At the east end, the main supporting pier could not be located in the river, and, on account of the steep bank, the end of the bridge could only have been located from 30 to 40 ft. back of the main pier, making it impossible to utilize an anchor span. This is also true to some extent at the west end. The anchors, therefore, would have had to be carried back some distance on private property, and would have been troublesome to locate. Moreover, the length of the main cable, in all, would have been fully twice the length of the chords of the arch.

Another element of economy for the arch was found in the possibility of obtaining, in the natural river banks, unlimited horizontal resistance for the thrust. The underlying gravel stratum was so hard that piles would probably have been unnecessary, but they were used as an additional precaution in case any future shifting of the current should tend to undermine the banks, which, nevertheless, were well protected with rip-rap. The economy of the foundations was very largely dependent upon the low spring of the arch, and this elevation was intentionally determined at such a level that extreme freshets would rise above the lowest steel, but that only on exceptional occasions the steel would be submerged. The reasoning that justified this decision was that an annual wetting could be no more harmful than ordinary weather conditions. It was thought by the writer that, of necessity, all ice would have passed out before the river could rise to the height of the steelwork, or, at least, that the ice would have become sufficiently broken up to be harmless. This opinion, it appears, was based upon insufficient knowledge of the location, for, during the first spring freshet which occurred after the bridge was built, the water rose to a level just above the main pin, to which the laterals are connected, and there was ice enough at the time to break a forging at one of the four corners.

To avoid future danger from this cause, the writer suggested that a reinforced concrete slab be constructed between the chords of the trusses, and extended to the level of extreme high water; and, as there remained a small, unexpended balance of the original appropriation,

Mr. Worcester. the towns favored the suggestion, and went so far as to put in the necessary steel beams for the support of the slab. This was not done, however, till late in the fall of 1905, and it did not seem wise to attempt to put in the concrete that winter. Forms for the concrete were designed, therefore, in such a way that they would form in themselves a substantial protection from the ice, and tend to allow it to slip by without jamming against the steelwork. The intention was to proceed with the concrete the next season, but, as it happened that the forms shielded the bridge efficiently without it, this work has not yet been put in, and the ice has passed out of the river for three years without further trouble. The wooden forms may be seen in the photographs of the finished bridge shown on Plate XXXVI.

While, therefore, it may justly be argued that it was a mistake to locate the spring of the arch so low, it was undoubtedly much more economical to do this and to provide a very thorough concrete protection for the exposed portions of the steelwork than it would have been to raise the skewback, thereby increasing enormously the cost of the foundations.

The appearance of the bridge has been referred to by many in complimentary terms, but, if it has a good appearance, it is due solely to the lines of strict utility being graceful. The only end in view in its design was to accomplish the object with the least possible expense. To the writer's mind, the drawing together of the chords at the end pins, however desirable from engineering considerations, is a serious blemish from an æsthetic standpoint. Moreover, the grade of the roadway, though not very noticeable in the photographs, is an unpleasant feature from a broadside viewpoint. To have made the most of the artistic possibilities, it would have been desirable to have added masonry towers over the spring of the arch, as in similar bridges across the Rhine; but such features, unfortunately, can rarely be afforded in the United States.

Mr. Snow. J. P. Snow, M. Am. Soc. C. E. (by letter).—When the writer was requested to confer with and advise the committee handling the Bellows Falls Arch Bridge, plans and bids, from various firms, for a bridge of two spans with the pier located so that there was a sharp angle between the lines of the spans, were at hand. It was apparent that, although the committee had called for bids on this lay-out, it had become convinced that an angle was objectionable; and, as the owners of the water-power objected to a pier on a direct line, there was no alternative but a single span. New bids were called for, under a general specification, as stated by the author, but, of the designs submitted, none that could be built for a sum less than one and one-half times the amount available was at all satisfactory.

Rather than accept an unsatisfactory design the members of the committee consented to the engagement of Mr. Worcester, who as-

sured them that a bridge of the arch type, satisfactory to the writer, could be built within the available means. Bidding plans and a specification were prepared, and the proposals justified Mr. Worcester's position. It is interesting to note that the bidders who had previously presented designs did not appear on the scene when a definite plan was issued. Mr. Snow.

The material was inspected by Henry B. Seaman, M. Am. Soc. C. E., and the field work by Mr. C. H. Restall, under the direction of the writer. The shop work was exceptionally good, and much credit is due to the contractors for their painstaking care, and to the erection force for their courage and resourcefulness in setting up the bridge under difficult conditions. The erection of bridges over New England rivers in the winter is hazardous; broadly speaking, it implies bad management somewhere. In the present instance, the error arose from the loss of time in the early part of the season by the committee while learning how to buy a bridge. No time was lost by the contractors after the work came into their hands.

The towns may well congratulate themselves on their bridge. It represents considerably more value than their payments covered.

F. W. Skinner, M. Am. Soc. C. E.—Everybody admires a great engineering work which is designed and built conservatively, thoroughly, slowly, and deliberately, sometimes even ponderously. Everything is done with great care and forethought, and with perfect apparatus. All the appliances are complete, and the entire construction is worked out in the most minute, most solid, and most monumental manner. Mr. Skinner.

There is no doubt that such works are great engineering triumphs, but, in achieving them, the engineer often deviates very little from established precedent, even though the construction is on a larger scale than usual, and the speaker thinks that such construction does not lead to professional progress, or at least to as great an extent as desirable.

All admire the great Forth Bridge, with its unprecedented span, but its weight and cost were great, and it took a long time to build. This is true of many other engineering works. The speaker has in mind an illustration, doubtless quite familiar to many, showing that extremely careful and costly engineering constructions are sometimes inadvisable, and by the use of more rapid, more daring and original methods, work can be executed which might otherwise fail.

In New York City, not very long ago, there was an important piece of engineering work, on which, before completion, radical repairs or changes were found to be necessary. An estimate was obtained, from an engineer eminent in that particular class of work, which involved the expenditure of somewhat more than $1 000 000. Eventually, the changes were effected for $100 000, by an entirely different method,

Mr. Skinner. devised by a member of this Society. This method was considered daring and impracticable until the contrary was demonstrated by its unqualified success. Therefore, considering the greatest good to humanity, the greatest advances seem to be made by, and the greatest praise to be due to, the engineer who accomplishes the best results, with the least money, with the greatest safety, and in the quickest time.

Judged by that standard, and by the results obtained, the designers and constructors of the Bellows Falls Arch Bridge rank very high in bridge building.

This span of 540 ft., with its substructure, cost only \$46 000, or less than \$3.50 per square foot of floor. Such a result is unprecedented for such a long span. The erection time is also unprecedented, for it required only 28 working days. In Europe a year or two would generally be taken in building a structure of that kind.

There are several features in the design of this arch which commend themselves to every bridge engineer. Among them, the scheme of making the crown connection with plates proved a very happy device, not usual in ordinary practice. The construction of the crown panel, too, is advantageous.

It is to be regretted that the author did not give the details of the members and connections, and it is hoped that he may yet present a paper which will deal with these features.

In the erection of the Bellows Falls Arch, a happy mean was established between a self-sustaining structure and a mass of falsework by giving it an economical amount of temporary support, and the result abundantly justified the means.

Another feature in the erection of this arch is its exemplification of the fast growing tendency to utilize the great advantages of steel derrick booms over other apparatus for handling heavy steel members. These booms were 60 ft. long, but Mr. Rights, if he were repeating the work to-day, might use 100-ft. booms, as he has in other recent erections.

The Bellows Falls work was admirably designed, and served its purpose thoroughly well. The highest compliment that can be paid to the bridge is to compare it with some of the notable bridges of similar type; because the methods used in their erection will show the excellence of this one much more effectively than any assertions.

The following description covers fairly well the erection features of all the large arch spans which have been built:

The famous Eads Bridge—"the Father of Arches"—had three spans, two of 537 ft. and one of 552 ft., which were noted as being the longest railroad spans, and for a very long time remained the longest arches. They were made with four ribs or trusses with steel-stave chords 18 in. in diameter, and were erected by the balanced, guyed, cantilever system. The trusses of adjacent spans were built

FIG. 1.—ERECTION OF THE EADS BRIDGE.

FIG. 2.—ERECTION OF THE WASHINGTON BRIDGE.

out simultaneously from the piers, and were supported by an elaborate system of guys or back-stays reaching from the successively erected panel points of the trusses back to the tops of pairs of falsework towers on the piers. Mr. Skinner.

Each tower was a pyramidal skeleton, 50 ft. high, with a 24 by 24-in. oak mast, 12 by 12-in. batter legs, and 18 by 24-in. oak sills set on special hydraulic jacks. The skewbacks were tied together by horizontal anchors through the piers, which made them self-sustaining for about one-quarter length, beyond which the two middle trusses were supported by eye-bar guys with sleeve-nut adjustments provided with very elaborate falsework supports, set on top of the trusses to diminish the deflection. The center panel connections were made by using the guy adjustments and by the operation of the jacks under the towers, which moved the latter vertically 6½ in. Both these means together, however, were not adequate to provide for extreme temperature variations, and there was great difficulty in making some of the connections. The chords were packed in ice, and special sections were cut to fit the last panels.

At the shore spans special anchorages had to be provided to secure the ends of the back-stays. One of them was made with castings set in a shaft excavated 30 ft. in solid rock, and the other with a horizontal oak girder, 4 ft. square, engaging a quadruple row of 12 by 12-in. sheet-piles driven in sand in the bottom of a deep excavation. All four ribs of each span were built simultaneously for the first three-elevenths of their length, after which, work on the outer ones was suspended until the center ones were completed. These then served as platforms from which the remaining outer ones were erected. The materials were put in place by a hand-power traveler, advanced by rack and pinion, and equipped with four derrick booms. The maximum clear height above high water is 73 ft. 9 in., and the cost, including that of the difficult substructure and approaches, was about $10 000 000.

Fig. 1, Plate XXXVIII, is to be valued for its historical associations. It shows the Niagara gorge with the three great types of long-span bridges, all built without falsework. In the foreground may be seen portions of the original railroad suspension bridge of 800 ft. span, with steel stiffening trusses which replaced the original trusses of wood and iron some 20 or 30 years after the construction of the bridge. In the background is the second cantilever built in America, the famous work of C. C. Schneider, Past-President, Am. Soc. C. E. Partly completed, and in the same plane as the suspension bridge, is the double-deck, 550-ft., spandrel-braced, arch span, designed by L. L. Buck, M. Am. Soc. C. E. This arch was also built by the guyed cantilever system. The trusses were assembled entirely by small overhead steel gantry travelers, which ran on the new top chords, outside

Mr. Skinner. of the old suspension bridge, and allowed the traffic to be maintained on both tracks of the latter while the new bridge was being built.

The adjustments for the connection of the center panel were made by slightly revolving the semi-arch trusses about their skewback pins by using a toggle inserted in a chain connecting each top chord with a temporary anchorage of I-beams bedded in concrete in chambers excavated for the purpose in the solid rock. The anchor chains were made with eye-bars proportioned for stresses of 1 000 000 lb. per truss, and were connected to parallelograms of eye-bars with vertical screw diagonals operated by sixteen men to a capstan-head, thus raising or lowering the span, as required.

The construction of the railroad arch was soon followed by that of another arch, for highway traffic, just below the Falls. It has a span of 840 ft., is about 200 ft. above the water, and was built in very much the same manner, but, as there was no horizontal top chord to form part of the anchorage, the guys for the cantilevered semi-trusses were lines of eye-bars attached to alternate panel points on the top chords as fast as they were built out. These were adjusted by the same toggle which had been used on the railroad arch.

This bridge was also built in the plane of an existing suspension bridge, but in this case the old structure was used for the support of the very light travelers which handled the members for the arch trusses, and as none of them weighed more than 5 tons, the load imposed on the old bridge was very small. The toggles were only required to lower the semi-arches, and this they accomplished with a force of twenty men on each. The bridge weighs 3 651 000 lb., and was erected by 100 men in about 3 months.

The Washington Highway Bridge across the Harlem River, New York City, has two 510-ft. main spans, each with six two-hinged plate-girder arch ribs of 90 ft. rise and 133 ft. clear height above high water. The ribs have a uniform depth of 13 ft., and the flanges are curved to parabolic arcs and support transverse bents, 15 ft. apart, carrying the floor platform. The spans were erected on framed-trestle falsework on piles, and materials were delivered from a service track at the skewback level, parallel to the bridge axis, to the erection travelers on the top flanges of the arch ribs. The travelers consisted of pairs of adjustable stiff-leg derricks which erected the six ribs simultaneously from both skewbacks to the crown and then moved back to the ends of the span on the permanent floor which they erected in advance. The channel span falsework had inclined bents providing an 80-ft. center opening for navigation. The ribs were swung by jack-screws on each falsework bent. Each span weighs about 1 670 tons.

The Victoria Bridge, across the Zambesi River, in Africa, is a double-track railroad structure with two-hinged spandrel-braced arch trusses of 500 ft. span, 400 ft. above the water. The connections are

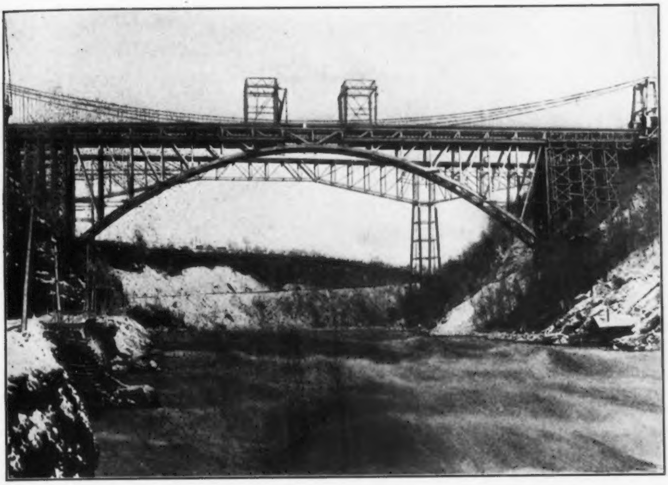


FIG. 1.—THE RAILROAD SUSPENSION, RAILROAD ARCH, AND CANTILEVER BRIDGES, AT NIAGARA.

FIG. 2.—THE HIGHWAY SUSPENSION BRIDGE AT NIAGARA BEING REPLACED BY A STEEL ARCH.

riveted, but, for erection, were provided with 2-in. auxiliary pins. The trusses are in battered planes, are 105 ft. deep at the skewbacks, and 15 ft. deep at the crown. Half of the 1 650 tons of steel in the bridge was carried across the gorge by a cableway of 10 tons capacity, with electric mechanism and an adjustable shear-leg tower. A line carried across the gorge by a rocket was the first step in the erection of the cableway. Mr. Skinner.

The arch trusses were erected, in a manner similar to that used in the Niagara Railroad Arch, as anchored cantilevers, fulcrumed on their skewback bearings, and the reactions were provided for by multiple anchor cables carried through U-shaped inclined tunnels in the solid rock, and adjusted by nuts and screws.

The erection travelers were simple platforms, moving on the horizontal top chords, and provided with 30-ft. derrick booms. To receive the last chord section, the center panel opening in the top chord was adjusted by hydraulic jacks, and the last bearing was made with planed shim plates. A safety net was at first swung under the travelers to protect the workmen, but was found to make them nervous. Contrary to American practice, the bridge was completely assembled in sections, at the shops in England where it was fabricated, thus involving considerable extra expense. The trusses were erected in about 5½ months.

The double-track, Müngsten, or Kaiser Wilhelm Bridge, across the Wupper River, in Prussia, is 350 ft. high, and has a clear span of 525 ft. The riveted trusses are battered 1 : 7, and were built as guyed cantilevers after the completion of the skewback towers erected on elaborate and heavy falsework, and the construction of the approach spans, erected on falsework trusses. These were assembled on the surface of the ground and hoisted bodily to position on top of the permanent viaduct towers, a proceeding which might apparently have been as well applied to the permanent spans, even as the towers might have served as their own falsework.

Both arch trusses and high-level roadway trusses, supported by spandrel posts on the arches, were erected simultaneously as cantilevers. Materials were delivered on a "low-level" service track on a falsework bridge, about at the skewback level, high above the surface of the water. The cable guys were adjusted by hydraulic rams, and hydraulic rams were inserted in the crown panel to release the lower chord skewback wedges which had been inserted to compensate for deflection. The crown joint was riveted, and then the final stresses were adjusted by hydraulic jacks at the skewbacks. The bridge weighs 5 622 tons, cost $1 230 000, and was erected in 22 months.

The single-track Garabit Viaduct, in France, has one two-hinged, 541-ft. arch span, 406 ft. high, with riveted trusses having a great rise and carrying spandrel towers which support the roadway trusses of

Mr. Skinner. about 125 ft. span. The skewback and viaduct towers were first erected, and then the approach spans were erected on shore, at both ends of the bridge, and launched forward by protrusion over the tops of the towers until they projected some distance beyond the arch abutments. Locomotive derricks were installed on them, and a cableway was set up with its towers on top of the skewback towers.

With these tools, the arch members, delivered on a low-level service bridge 100 ft. above the water, were erected, the two end panels on each side being assembled on falsework, while the remainder was built out simultaneously from each abutment as cantilevers, guyed to the tops of the towers by 28 steel cables. The other roadway spans were erected simultaneously as cantilevers. The erection lasted about 4 years.

The four 200-ft. spans of the electric car bridge across the Schuylkill River, in Fairmount Park, Philadelphia, each have three spandrel-braced riveted trusses, and were erected with the lower chords supported on pile falsework. Materials were delivered on a track laid on the bridge floor, and were handled by a wooden gantry traveler, 72 ft. high, with a 23-ft. overhang.

The Rochester Driving Park highway bridge across the Genesee River, has two three-hinged, spandrel-braced arch trusses of 428 ft. span which were erected on unusually heavy framed falsework more than 212 ft. high, with a wide trussed opening over the river. The truss members, having a maximum weight of 10 tons, were assembled by a 16 by 30-ft. wooden tower traveler, 28 ft. high, with a derrick boom on each corner. The falsework was notable for its great strength and rigidity, equal to many permanent wooden trestle viaducts for railroad service.

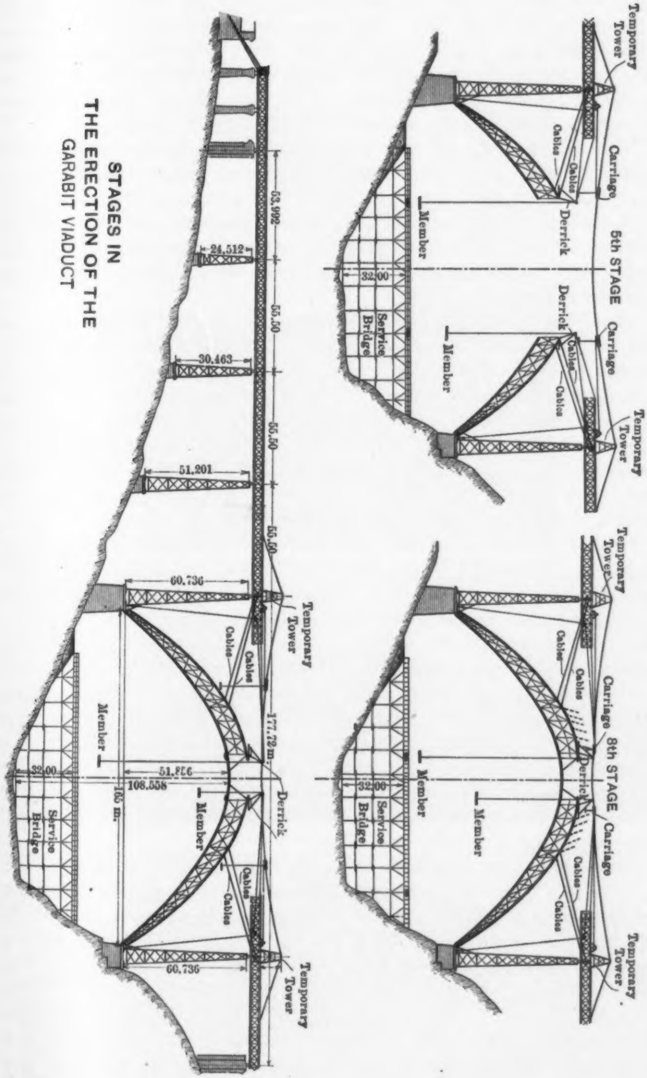
The longest arch span in Europe is that of the Bonn Bridge, 614 ft., which was erected by 8-ton electric gantries on 7-story falsework, 112 ft. high, with two trussed openings of 102 ft. for navigation. Upper falsework, long since obsolete in America, was built above the curved bottom chords to provide a horizontal track at the top chord level for the two 8-ton electric gantries by which materials were hoisted from boats and erected. The 594½-ft. arch span of the Dusseldorf Bridge was erected in a similar manner, its falsework being provided with a 164-ft. trussed opening for navigation.

The Lake Street highway bridge, across the Mississippi River at Minneapolis, has two 458-ft. spandrel-braced arch spans. These were erected on falsework 120 ft. high, the piles having been driven through the ice. The falsework terminated at the curved lower chord, and the superstructure was erected by an overhead timber tower traveler with hoisting tackles suspended from an overhang, which traveled on the finished deck of the bridge. This method necessitated the unusual procedure of erecting the arch trusses from one abutment across the

FIG 1.—THE MUNGSTEN, OR KAISER WILHELM BRIDGE.

FIG. 2.—ERECTION OF THE MUNGSTEN, OR KAISER WILHELM BRIDGE.

Mr. Skinner.



STAGES IN THE ERECTION OF THE GARABIT VIADUCT

Fig. 3.

Mr. Skinner. entire span to the other abutment and making the final connection at or near the skewback pin. This was accomplished successfully, and without difficulty in adjusting the last panel members. During erection, the unbalanced longitudinal thrust, due to unsymmetrical loading on the falsework, was provided for by very heavy inclined timbers bracing the falsework bents diagonally from top to bottom.

The 377-ft. span of the Kornhaus Bridge, in Switzerland, was erected on high and very expensive falsework supporting the arched lower chords on a solid convex plank floor platform, like the lagging for a masonry arch, above which upper falsework was built for the erection of the horizontal roadway trusses supported on spandrel towers, and for light gantry traveler and material tracks outside the arch trusses.

The Panther Hollow Bridge, in Schenley Park, Pittsburg, has one span with four 360-ft., three-hinged spandrel-braced arch trusses. One peculiarity of this erection was that the trusses were erected from one abutment to the crown before the falsework for the other half of the span was built. Field connections were made with small pins at panel points, and, after the arch was swung and these connections had adjusted themselves to the dead-load stresses, the joints were all field-riveted, on the assumption that the rivets would carry the live-load stresses of the bridge in service.

One of the most elaborate of arch span erections was that of the Alexander III Bridge across the Seine, Paris, which has fifteen cast-steel segmental ribs of very flat curvature. During erection these were suspended from a movable overhead falsework span, mounted on towers traveling transversely to the bridge axis. The falsework span was assembled on shore and erected by protrusion across the river, with an auxiliary emergency scow under the forward end. The span was traversed by a pair of trolleys which took the arch segments from shore and sustained them until they were assembled to the preceding ones and were supported by temporary suspension from the trusses. After a pair of ribs was thus simultaneously erected, the pair was swung by slacking off the suspension, and the traveler moved two panels forward and erected the next pair, and so on.

These examples illustrate the principal types of long-span arch erection, and describe most of the principal structures thus far built, giving a general idea of their structural characteristics and of the time and cost of erection. Only two or three of them have spans exceeding that of the Bellows Falls Bridge, and certainly none of them was erected with anything like its economy, or with as small a force as 36 men, or in so short a time as 28 days. These figures and the total cost, contrasted with those of the other bridges, pay a higher tribute to the skill and courage of the designer and erector than any mere compliment or admiring criticism.

FIG. 1.—ERECTION OF THE GARABIT VIADUCT.

FIG. 2.—ERECTION OF THE BONN BRIDGE.

FIG. 3.—ERECTION OF THE DÜSSELDORF BRIDGE.

Mr. Skinner.

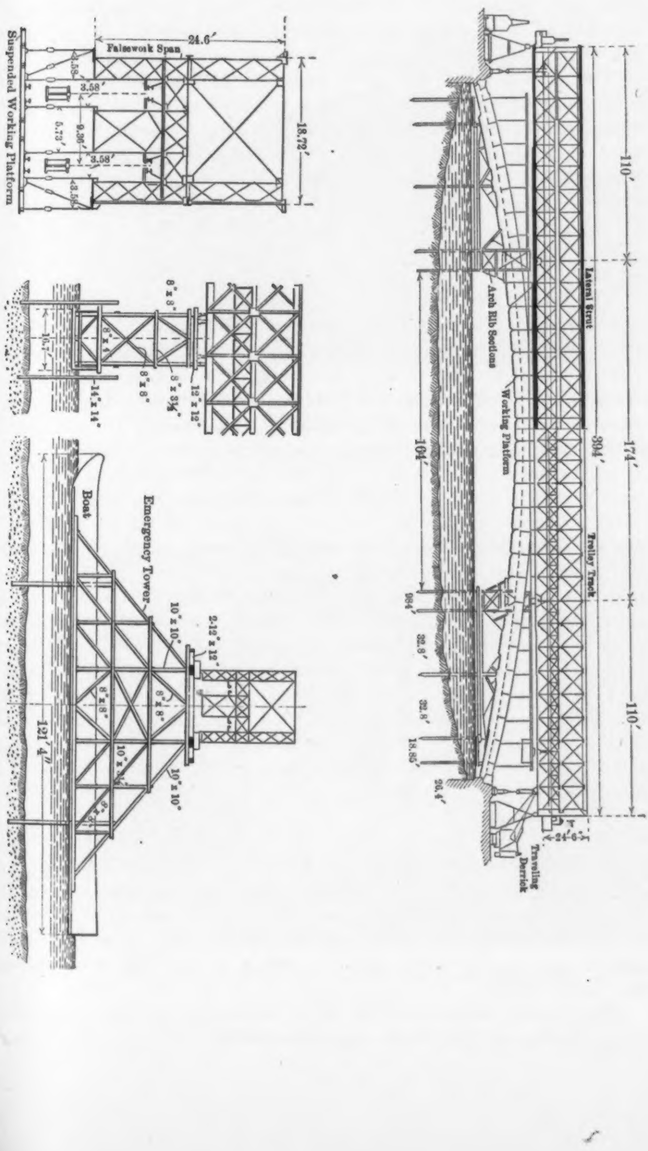


FIG. 4.

Mr. Quimby. HENRY H. QUIMBY, M. AM. SOC. C. E.—The method of erection described in the paper is interesting, and the celerity with which the work was accomplished is remarkable. Good fortune also seems to have attended the operation, in the continued stability of the ice, for, if a thaw had occurred, it might have caused a serious disaster.

The statement of the settlement of the falsework suggests the question whether any of it was due to overloading the blocking. To what unit pressure was the wood subjected? A recent case of settlement of falsework was found to have been due to the fact that the unit pressure on the wood was sufficient to cause it to yield slowly and keep on yielding under continued load, although well within what was regarded as a safe stress. Saturation by rain may soften the wood enough to permit compression which subsequent drying out will not recover.

The means used for securing proper bearing in the joints of the arch are not mentioned in the paper. It must have been found, however, that some of the joints did not meet truly. Was any fitting required, and how was it accomplished? The author has mentioned some lining or shimming, but it is not clear whether this referred to some of the joints or to the packing at the crown. The distribution of the stresses in an articulated arch will be materially affected by the fit of the members. Even if the chords be brought to a bearing by some fitting of the connections of the diagonals, the chord joints are likely to cause trouble by being out of true—the faced ends not in contact throughout—unless the splices are made of the full strength, and such splices are generally clumsy and expensive. When a box-chord in compression transmits its stress through a faced joint, and the contact is at one edge only, that edge or flange is overloaded, and if the eccentricity be sufficient to concentrate the whole transmitted force on one laced flange, the resistance of the lacing there may be overcome entirely, and then the burden will fall on the other and disaster ensue. It seems probable that this was the process of the failure of the Quebec bridge members, whether the eccentric joints were due to errors in the angle of facing or the unavoidable camber provision for relative changes of direction of members in the progressive deflection.

The design of the abutments, though not forming a part of the subject-matter of the paper, is referred to in Mr. Worcester's communication, and as he states that the gravel bottom was so hard that piles would probably not have been required, it becomes of interest to learn how far the piles that were used were driven into that gravel, and by what method, and what proportion of the load it was thought they could bear.

The avowed purpose of the detail at the crown—the maintenance of parallelism in the chords—is commendable, as is also the character

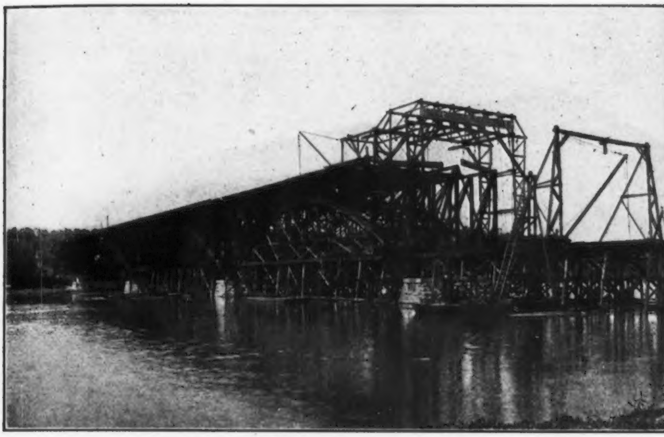


FIG. 1.—THE ERECTION OF THE FAIRMOUNT PARK BRIDGE, OVER THE SCHUYKILL RIVER, IN PHILADELPHIA.

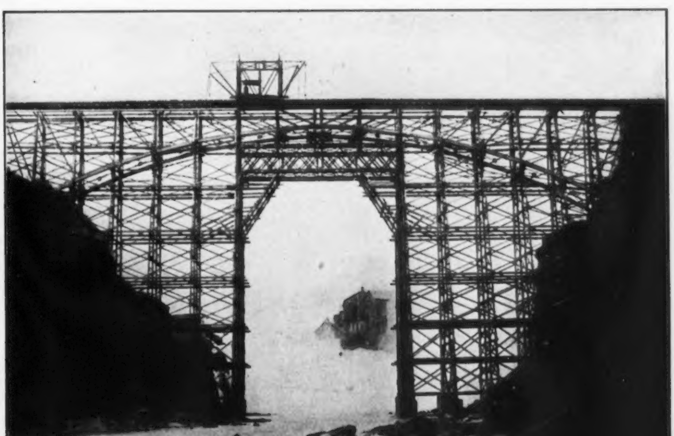


FIG. 2.—THE ERECTION OF THE ROCHESTER DRIVING PARK BRIDGE OVER THE GENESEE RIVER.

of the detail. If the same principle had been applied to the end hinges also, the appearance of the structure would have been more graceful, at least to the miscellaneous eye. This is the plan that was followed in the arches of the Washington Bridge, over the Harlem River, New York City, where the fact of the use of end hinges was concealed by continuing the full depth of ring close to the skewback and stiffening the web for the pin hinge bearing. If the fit of this crown key is good, the detail is better than an absolute pivot, such as a pin is intended to be, because it affords facility for some shift of the line of pressure which will be an accommodation to the stresses in the arch members without any appreciable increase of stress from temperature changes. Mr. Quimby.

The statement in the paper that the Bellows Falls Bridge is the only example in the United States of a steel arch with a suspended floor, calls for a note of the fact that there is at least one other—a hingeless arch of 82 ft. span, with a suspended reinforced concrete floor, in Philadelphia, on the line of Duval Street, over the Pennsylvania Railroad. The conditions required a very thin floor, and the situation made it desirable that the bridge should be graceful in appearance. The arch has fixed ends and a rigid though shallow crown. No effects of temperature changes can be observed in the structure, in either hot or cold weather. The adjustment to fit the abutments was accomplished by chipping the concrete bearings at one end until the joints matched.

Philip Aylett, Assoc. M. Am. Soc. C. E. (by letter).—Great credit is due the author and his associates for the ingenuity and skill they displayed in the erection of this bridge. Judging from the designs and photographs, the structure seems to be admirably adapted to the location. It is also pleasing to know that, contrary to general procedure, the beautiful stream which it spans has been left free from obstruction by intermediate piers and rip-rap. Mr. Aylett.

Although this structure has not the weight of metal possessed by some of its predecessors of similar type in Europe and America, and is not as important as a thoroughfare, still none of these bridges compares favorably with it in ingenuity and in economy of design and erection, and it stands well to the front as a clean-cut and representative example of "the art of doing that well with one dollar, which any bungler can do with two after a fashion."

It is refreshing to note, also, that, in several features, there have been radical departures from old methods, and this structure in many ways will stand alone and afford precedent for similar undertakings.

In view of the depth (25 ft.) and character of the stream, the hardness of the bottom, the small penetration of piles, and the great height of the superimposed truss weight above the bed of the stream (125 ft.) —the risk from ice and floods involved in the method of erection was

Mr. Aylett. very great, and was doubtless seriously considered from every possible point of view by the designers prior to adoption.

The latitude of the site and the continued severe temperature were in all probability factors which entered largely into the successful results achieved. On account of the frequency of freshets and the enormous quantities of drift, the method of erection and type of structure of such span length would hardly be considered by most engineers in the South or West.

The author states that the piles for the falsework were cut off and capped 3 ft. above the low-water line. This elevation was doubtless chosen as being the most convenient, and as providing a more rigid support for the superimposed towers, track, etc. There is also another consideration, which should never be overlooked in the design and construction of trestles and falsework over dangerous streams: It is noted that the longitudinal stringers for the service track were laid directly upon the pile caps, thus bringing this track about 4 or 5 ft. above low water. In some streams this elevation above low water has been found to be the plane of maximum drift during freshets. In all streams to be trestled, or in which important falsework is to be placed, it is a good safeguard to ascertain the elevation of the plane of maximum drift, and thus fix the elevation of all longitudinal members of the structure—above or below this plane—so that they may offer the least resistance to the drift.

This was doubtless considered at Bellows Falls, and as the elevations of the longitudinal stringers and braces were probably located with high-water and ice requirements in view, a higher elevation of the service track was unnecessary. It would have been well to have broken the continuity of the service track at its junction with the towers so that the intermediate portions, between the towers, could have given way and floated off down stream under the pressure and impact of drift or ice, thus leaving the towers intact and free from strains which might arise from the accumulation of drift against the intermediate portions of the service track. This is usually a wise precaution, as sufficient time ahead of a rising flood (especially at night or during bad weather) is not always afforded to prepare for them.

In adopting this design and method of erection, in the face of many unfavorable conditions and hazards, the designers and erectors have not only shown themselves to be possessed of ingenuity and skill, but also daring, backbone and determination—factors which have always entered and always will enter into great achievements and engineering triumphs, everywhere.

Mr. Rights LEWIS D. RIGHTS, ASSOC. M. AM. SOC. C. E. (by letter).—This paper was presented more as a record of work done than as a subject for discussion, and therefore the writer is pleased to find that it has attracted attention, and is grateful for the kind expressions of appreciation.

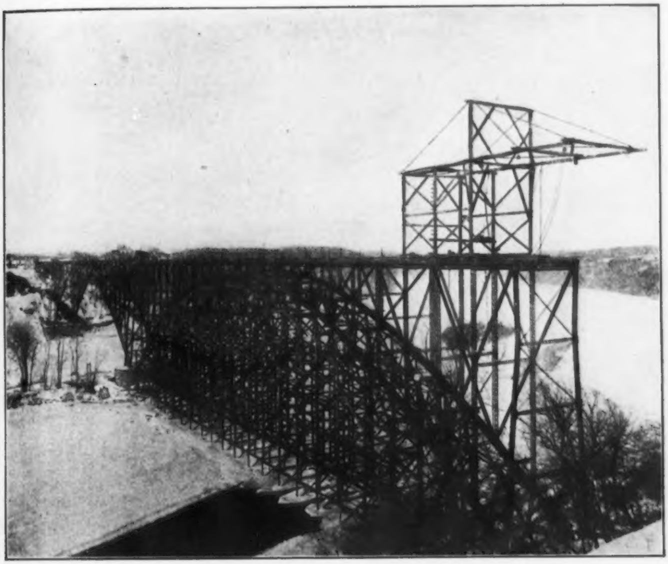


FIG. 1.—THE ERECTION OF THE LAKE STREET BRIDGE, OVER THE MISSISSIPPI RIVER, AT MINNEAPOLIS.

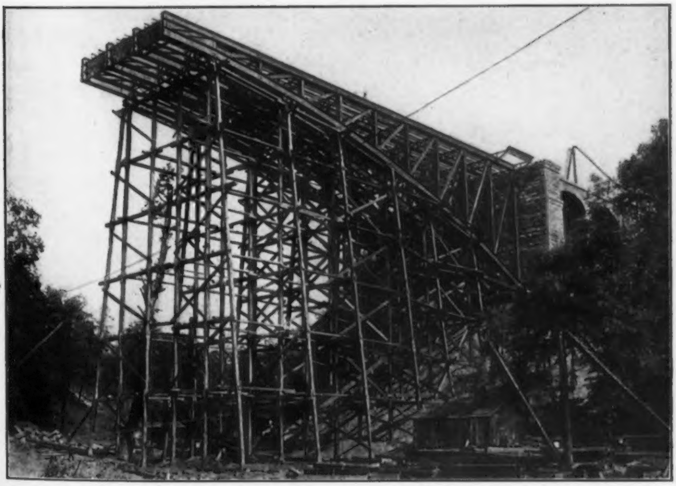


FIG. 2.—THE ERECTION OF A HALF SPAN OF THE PANTHER HOLLOW BRIDGE.

Messrs. Worcester and Snow have both been very kind in their compliments on the work of erection, but the writer feels that modesty has led them to take insufficient credit to themselves for the bold design and carefully worked out plans. The contractors owe much to these gentlemen for their continued assistance and support in a new and complicated problem. Several members have expressed a desire to see more complete details of the center hinge at the crown of the arch. The writer wishes to make up for this omission by submitting Fig. 5, a general plan showing the center and adjoining panels. This shows the total clearance of 4 in. between the chords and the theoretical opening between the center struts, and, with what has already been stated in the paper, will not require further explanation. Mr. Rights.

Mr. Quimby asks some questions which are interesting, even though they are not connected with the work of erection.

Regarding the piles in the foundations, the writer understands that, when the foundation pits were excavated, both Mr. Worcester and Mr. Snow inspected the bottoms very carefully. Mr. Worcester stated that he would not hesitate to put a load of from 5 to 6 tons per sq. ft. on the soil, which was fine gravel, very solidly compacted with sand. The cost of driving the piles, however, was a very small item compared with the total cost of the bridge, and, as the banks are an alluvial deposit, it was thought best to use piles to cover the possibility of a soft stratum existing below the point reached by the sounding rod, as well as to provide temporary safety in case of unexpected scour. The contractors for the substructure are dock builders, and carry on a large business in the vicinity of Boston; therefore they were well equipped with all kinds of pile-drivers. They rented a large scow, owned by the paper company, and on this rigged leads, an engine, and an 1 800-lb. drop hammer. Piles were driven to refusal in the bottoms of the pits, with a penetration of from 7 to 10 ft.

Regarding the settlement of the falsework, the writer is convinced that this was mainly due to the piles. His experience has shown that settlements in framed bents are caused by the shrinkage or contraction of the sills rather than the posts, and he has adopted a maximum of 400 lb. per sq. in. for the loading of the average yellow-pine falsework. For use in computation, this same maximum stress is taken for posts and sills, as in general it is not feasible to provide iron caps to spread the bearing on sills. The load on the *F* towers from the steelwork was only about 200 lb., and, after the wind pressure on the leeward side of the tower was added, it was found to be less than 400 lb. Of course, some settlement is always expected in erection work; consequently, erectors have learned to wedge up somewhat higher than the actual camber requirements.

Regarding the bearing of the faced joints of the chords of the arch, these were calculated along theoretical lines, no provision being

Mr. Rights. made for camber, and each bevel was computed from both the line of the tangent to the joint and the chord between panel points. The angles for all these bevels were shown to minutes and seconds, and the bevels were given to $\frac{1}{32}$ in. Much care was taken in the shop to see that the bevels were carefully milled to the exact angle. The day after the arch was swung off, the writer went over it very carefully. At that time, the chords had not been lined up, and four or five of the joints showed openings of from $\frac{1}{16}$ to $\frac{3}{32}$ in. on the channel flanges. These discrepancies were not confined to the upper or lower flanges, but were found to occur in different joints at either top or bottom. Mr. C. H. Restall, the Field Inspector, suggested that these discrepancies could be remedied by the use of very thin steel wedges, which could be driven into the joint so as to insure a bearing before the load of the floor came on the structure. As the lateral rods had not been adjusted, Mr. Westbrook, the Superintendent, and the writer felt that the discrepancies would adjust themselves as soon as the chords were properly lined up. The writer submitted this view to Mr. Snow, who went over the matter with Mr. Restall, and came to the same conclusion. This assumption happily proved to be the case, and, after the chords were properly lined, Mr. Restall was fully satisfied that the joints matched, and that the bearing was distributed throughout the whole depth of the chord. Several months after the bridge was completed, the writer again inspected it, taking special care to examine the joints on which he had noticed the discrepancies. He scraped away the paint with a knife, but could not discover any crack. It might have been possible that the top of the chord was taking the full load, while the bottom had just begun to bear; but, as all the joints are at the panel points, and are well riveted, such a possible discrepancy is probably fairly well provided for.

The writer is glad to learn that this paper has brought out the record of another arch, with suspended floor, even though of considerably shorter span.

Mr. Aylett's suggestion in regard to breaking the continuity of the service track at the junction with the towers is a good one. The ordinary variation of the Connecticut River, however, is not as great as in the rivers of the South and West, and ample notice of any marked rise is generally given. The elevation of the center line of the end pins was placed at Elevation 89.6, or 10 ft. above the level of the top of the dam, and the high-water elevation is 97.2. The piles were capped at the level stated principally on account of convenience, although it was felt that if a freshet should take place, the ice and drift would pass out above the level of the service track. This is confirmed by Mr. Worcester's discussion, in which he states that during a spring freshet the ice broke a forging just above the level of the end pins.

FIG. 1.—ERECTION OF THE KORNHAUS BRIDGE.

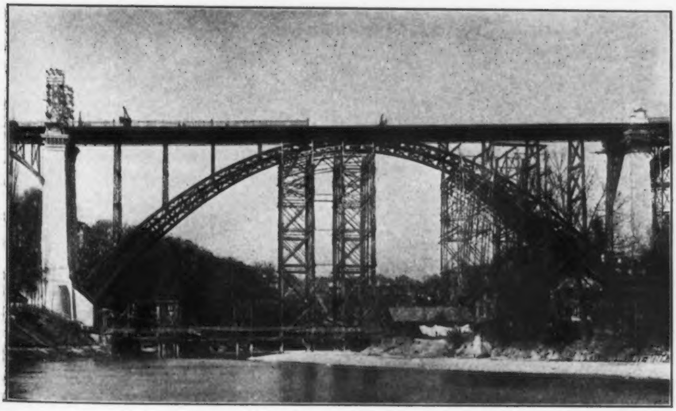


FIG. 2.—ERECTION OF THE KORNHAUS BRIDGE.

Mr. Rights.

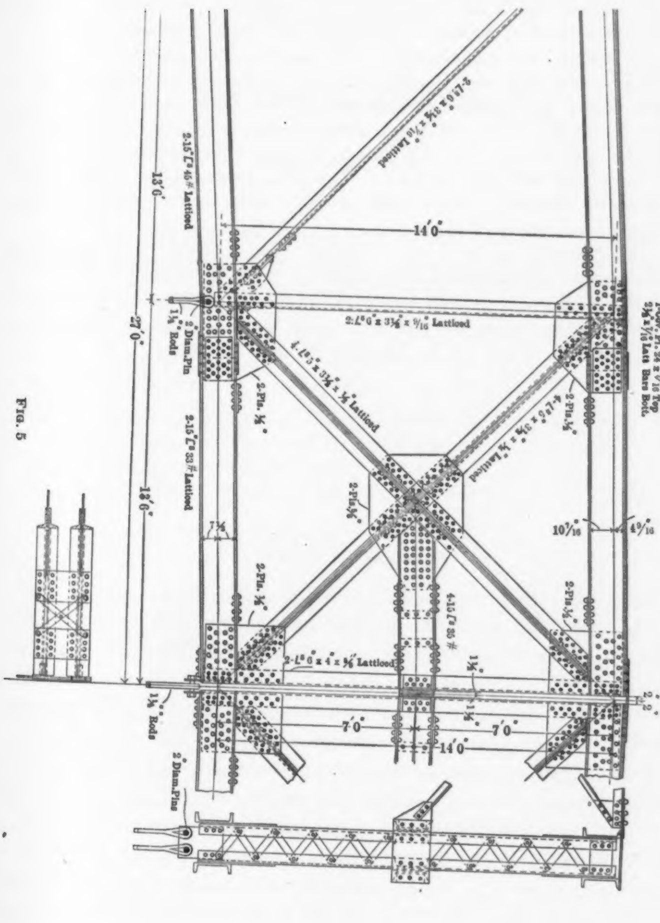


FIG. 5

Mr. Rights. The writer appreciates Mr. Skinner's comparisons with the arches of the United States and Europe. Mr. Skinner brings out an important point regarding the development of the erection of steelwork in America, which has appealed very forcibly to the writer. The last ten years have witnessed a wonderful change in methods. Booms are made longer, and engines and tackle are heavier. The use of steel for erection travelers and derricks has become quite common, as it has been found to be both stronger and lighter than wood. To-day, very few pieces are lifted by hand, power being relied on to take care of all the operations. For this reason, a comparison between the erection of the newer and older arches can hardly be considered as such, and Mr. Skinner's discussion represents an interesting record of the development of modern methods.

The writer has the faith to believe that bolder schemes and larger enterprises will be carried forward in the next decade, and, if any of the data given in this paper are of use to engineers carrying forward these schemes, he will feel many times repaid for his work in preparing it.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1084.

THE FLOOD OF MARCH, 1907, IN THE SACRAMENTO AND SAN JOAQUIN RIVER BASINS, CALIFORNIA.*

By W. B. Clapp, M. Am. Soc. C. E., E. C. Murphy, M. Am. Soc. C. E., and W. F. Martin, Jun. Am. Soc. C. E.

With Discussion by Messrs. C. E. Grunsky, H. M. Chittenden, H. F. Labelle, Luther Wagoner, H. H. Wadsworth, George L. Dillman, Edwin Duryea, Jr., and W. B. Clapp, E. C. Murphy, and W. F. Martin.

Introduction.

The Sacramento and San Joaquin Valleys were visited, in March, 1907, by one of the most destructive floods that have ever occurred in California, the resulting financial loss being unquestionably greater than that from any other flood of which there is record. The greatest damage was done in the valleys of the trunk streams, especially Sacramento Valley. The Lower Sacramento River and its two largest tributaries, Feather and American Rivers, reached the highest stages ever recorded, and record stages were reached by other tributaries of the Sacramento and by the San Joaquin and its tributaries.

The flood was remarkable in many respects. In the first place, it was preceded by a period of heavy precipitation, and consequent flood stages of all the streams, a condition which had prevailed intermittently

* The data upon which it is based were collected by the Water Resources Branch of the United States Geological Survey in co-operation with the State of California, and the paper is published by permission of the Director of the Survey. This paper was prepared under the direction of W. B. Clapp, District Engineer, by E. C. Murphy and W. F. Martin, Engineers.

Further acknowledgments are due to Mr. J. H. Scarr, the district forecaster of the United States Weather Bureau, and to the engineering department of the Southern Pacific Railway, for data furnished.

for several preceding weeks. As a result, the earth was thoroughly saturated, and all the surface basins which impound and store flood waters temporarily were full. Particularly was this true of the large flood basins on each side of the Sacramento River. Then, too, this flood was due to a general precipitation of extraordinary intensity throughout the entire drainage basin (the storm covering a period of several consecutive days), and also to comparatively high temperature and consequent rapid melting of snow in the higher altitudes.

This flood was remarkable, also, because of the record-breaking stages of so many of the streams, such as the Lower Sacramento River and the Feather, Yuba, and American Rivers. Not only were they higher than ever known before, but they maintained their high stages for a moderately long period. All the other streams of the water-shed also maintained high stages for a like period, so that the resultant was a flood of exceptional height and extent, and of considerable duration. For the 4-day period, March 18th to 21st, the mean rate of run-off from the mountains and foot-hills of the Sacramento Basin alone was about 530 000 cu. ft. per sec., or more than 22 cu. ft. per sec. per sq. mile.

During this flood, special effort was made by the engineers of the United States Geological Survey to obtain valuable flood data. The flow of nearly all the important tributaries of both the Sacramento and San Joaquin River systems was gauged in the foot-hills above the point of débouchure. The flow from 83% of the mountains and foot-hills in the Sacramento Basin was measured at eleven gauging stations. In the San Joaquin Basin the flow from 41% of the mountains and foot-hills was measured at six gauging stations. Unfortunately, no gaugings were made of the San Joaquin itself.

It is believed that the data obtained during this flood will fully repay the State of California for its generous co-operation with the United States Geological Survey in the study of its water resources. Data are now available for planning for these great valleys a more comprehensive reclamation system than has been possible heretofore. The importance of the data collected will be appreciated when it is recalled that the rate of run-off from the mountains and foot-hills of the Sacramento Basin alone for a period of 4 consecutive days, March 18th to 21st, was 112% greater than the rate used as a maximum by the 1904 Commission of Engineers, after a careful study of all flood

data on record, including those of the 1904 flood. It is doubtful if any combination of causes or conditions will ever produce a larger rate of delivery of water to this valley for a 4-day period than occurred during the flood of March, 1907.

Topography and Drainage of the Water-shed.

California is traversed, in a general northwest-southeast direction, by two distinct and approximately parallel ranges of mountains which extend almost the entire length of the State. Near the eastern border is the Sierra Nevada; not far from the shore line on the west is the Coast Range. These two ranges merge into each other about 40 miles south of the California-Oregon boundary line, the meeting point being Mount Shasta, which has an elevation of 14 380 ft. They are merged again south of Bakersfield by a cross-range known as Tehachapi Mountains.

The elevation of the Sierra Nevada ranges from about 6 000 ft. east of Mount Shasta at the north, to 14 501 ft. south of Yosemite National Park where the range culminates in Mount Whitney. Beckwith Pass, about 150 miles south of the northern boundary line, is the lowest pass through the range, and has an elevation of 5 300 ft. The Coast Range is comparatively low, and is unbroken except at Carquinez Strait and the Golden Gate which permit the drainage through Suisun Bay to reach the Pacific.

The Sierra Nevada and Coast Ranges, merging at the north and south, inclose a water-shed approximately 58 000 sq. miles in area, with a single outlet near the middle of the western side. This water-shed is somewhat elliptical in shape, and has a length of about 540 miles from north to south and a width varying from 120 to 150 miles. It is drained by two large river systems, the Sacramento in the north and the San Joaquin in the south, and these are quite commonly referred to as the Sacramento and the San Joaquin River Basins.

Sacramento River has its source in the region of Mount Shasta, and flows almost due south through the trough of the water-shed until it discharges into Suisun Bay. The San Joaquin rises in the Sierra Nevada, in the region of Mount Lyell, just east of Yosemite National Park, at an elevation of 13 000 ft., and flows southwestward until it emerges from the foot-hills into the trough of the valley, when it turns and flows northwestward to its junction with the Sacramento near

Suisun Bay, through which the combined volume of the two systems finds an outlet to the Pacific by way of San Pablo and San Francisco Bays and the Golden Gate.

The drainage of the water-shed determines its division into three distinct basins: On the north is the Sacramento Basin, 27 100 sq. miles in area, drained by the Sacramento River and its tributaries; in the center is the San Joaquin Basin, about 18 300 sq. miles in area, drained by the San Joaquin and its tributaries (excluding Kings River, which, for reasons given later, is classified under the Lake Basin); in the south is the Lake Basin, with an area of about 12 600 sq. miles, containing several lakes with their tributary drainage, but at the present time having no outlet discharging to the sea.

That portion of the three basins which is inclosed by the sharply-defined line of the foot-hills is called the "Great Valley of California." This valley has a length of about 400 miles from north to south, an average width of about 40 miles, and an area of probably 15 000 sq. miles, and is surrounded by steep mountains. The western mountain slope—that of the Coast Range—is comparatively narrow, having an average width of about 18 miles. Considering the entire length of the district, from north to south, the precipitation, as a whole, is light, and perennial streams are few, but, in the region about Clear Lake and Mount St. Helena, in the Lower Sacramento Basin, the precipitation is remarkably heavy, and occurs almost entirely as rain. The eastern mountain slope, which has an average width of about 58 miles, is visited by rather heavy precipitation throughout almost its entire length from north to south, particularly in the central part of the Sacramento Basin. A large percentage of the precipitation occurs as snow on the higher elevations. From this slope come all the larger tributaries to the Sacramento and San Joaquin Rivers and the San Joaquin itself, as well as the principal tributaries to the Lake Basin. The change from mountain to valley is quite abrupt along a well-defined line, but the slope of the valley is gentle and uniform.

What is commonly called the Sacramento Valley extends northward only to Iron Canyon, near Red Bluff. In the Report of the Commissioner of Public Works to the Governor of California, in 1894 (page 28), the valley is described as having a total area of about 4 250 sq. miles, divided as follows: 2 510 sq. miles of high lands, not subject to overflow; 450 sq. miles of lower lands, overflowed occasionally

MAP OF WATER-SHED OF SACRAMENTO AND SAN JOAQUIN RIVERS IN CALIFORNIA SHOWING TRIBUTARY STREAMS, UNITED STATES GAUGING AND PRECIPITATION STATIONS, AND PROPOSED RESERVOIR SITES IN SACRAMENTO BASIN

Fig. 1.

by high floods; 1 250 sq. miles of low lands, overflowed periodically; and 38 sq. miles of perennial stream surface. Below the mouth of Stony Creek (Plate XLIV and Fig. 1) the central portion of the valley is a flood plane of unusual extent, the immediate river banks being from 5 to 20 ft. higher than the land on either side some distance from the river. In the vicinity of the river banks the ground slopes rapidly from the river toward the trough of the flood basins on either side, but, as the bottom of the trough is approached, the slope becomes more gradual. The lowest portions of the flood-basin troughs are from 2 to 7 miles from the river channel.

The large flood basin on the west side of the Sacramento is divided into two smaller basins by a ridge of débris brought down by Cache Creek. These are the Colusa Basin in the north and the Yolo Basin in the south. The large flood basin on the east side of the Sacramento is divided into four smaller basins by Marysville Buttes and the Feather and American Rivers. From north to south, they are called Butte Basin, Sutter Basin, American Flood Basin, and Sacramento Flood Basin. Fig. 1 shows the position of the flood basins. The following data regarding the area and capacity of these smaller flood basins are taken from the Report of the Commissioner of Public Works to the Governor of California, for 1894:

Colusa Basin is 50 miles long, from 2 to 7 miles wide, and has a capacity of 690 000 acre-ft. at flood stage. It discharges into Sacramento River above Knight's Landing through Sycamore Slough.

Yolo Basin has a length of 40 miles, an average width of 7 miles, and a capacity of 1 115 000 acre-ft. at flood stage. It discharges through Cache Slough into Steamboat Slough, and thence into the Sacramento near the foot of Grand Island, about 25 miles above the head of Suisun Bay.

Butte Basin is north of Marysville Buttes, and has an area of from 30 to 150 sq. miles, depending upon the river stage, and a capacity of 460 000 acre-ft. at flood stage. It discharges through Butte Slough into Sutter Basin.

Sutter Basin is south of Marysville Buttes and north of the Feather River. It has an area of 138 sq. miles, and a capacity of 895 000 acre-ft. at flood stage. It discharges through sloughs into Sacramento River above the mouth of Feather River.

The American Flood Basin is south of Feather River and north of

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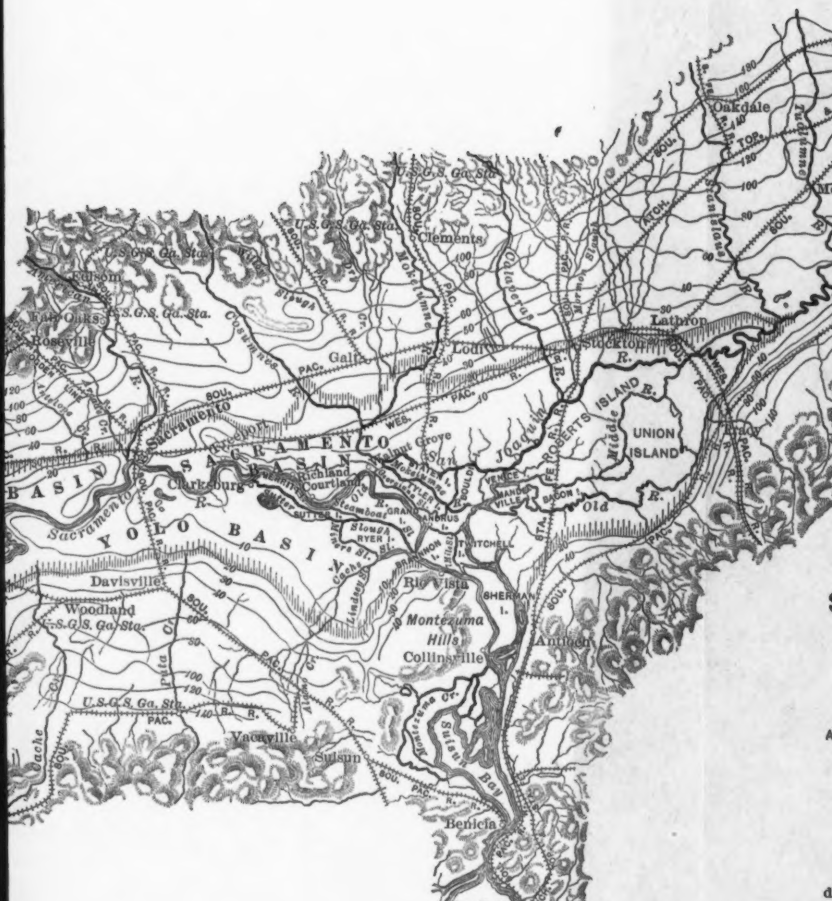
surface in the river.
BUTTE BASIN
COLUSA BASIN
SUTTER BASIN
AMERICAN BASIN
YOLO BASIN
SACRAMENTO BASIN
UNION ISLAND
ROBERTS ISLAND
Marysville Buttes
Merced
Oakdale
Modesto
Newman
Westley
Lathrop
Stockton
Lodi
Galt
Clements
Folsom
Fair Oaks
Roseville
Wheatland
Marysville
Yuba City
Oroville
Palermo
Biggs
Durham
Chico
Grimes
Colusa
Princeton
Jacinto
Munroeville
Orland
Willows
Williams
Dunnigan
Knights Landing
Woodland
Davisville
Sacramento
Clarksburg
Richland
Courtland
Walnut Grove
Rio Vista
Montezuma Hills
Collinsville
Antioch
Vacaville
Suisun
Benicia
San Pablo Bay
Suisun Bay
Feather R.
Sacramento R.
San Joaquin R.
Cosumnes
Mokelumne
Calaveras
Stanislaus
Tuolumne
Merced R.
Middle R.
Old R.
Cache
Putah
Butte Cr.
Dry Cr.
Stony Cr.
U.S.G.S. Ga. Sta.



Oakdale
Tuolumne
Stanislaus
Clements
U.S.G.S. Ga. Sta.
Mokelumne
Calaveras
Mormon Slough
Folsom
American
Fair Oaks
Roseville
Cosumnes
Dry Cr.
Lodi
Galt
Stockton
Lathrop
Sacramento
Freeport
SACRAMENTO BASIN
Walnut Grove
San Joaquin
ROBERTS ISLAND
UNION ISLAND
Middle
Clarksburg
Richland
Courtland
Steamboat Slough
Old R.
YOLO BASIN
Davisville
Woodland
Putah Cr.
Cache Sl.
Lindsey Sl.
Rio Vista
SHERMAN I.
Montezuma Hills
Collinsville
Antioch
Montezuma Cr.
Suisun Bay
Vacaville
Suisun
Benicia
San Pablo Bay
TWITCHELL I.
BRANNON I.
RYER I.
GRAND I.
ANDRUS I.
TYLER I.
STATEN I.
BOULDIN I.
VENICE I.
MANDEVILLE I.
BACON I.
SUTTER I.
MERRITT I.
SOU. PAC. R. R.
WES. PAC.
A. T. & S. FE. R. R.

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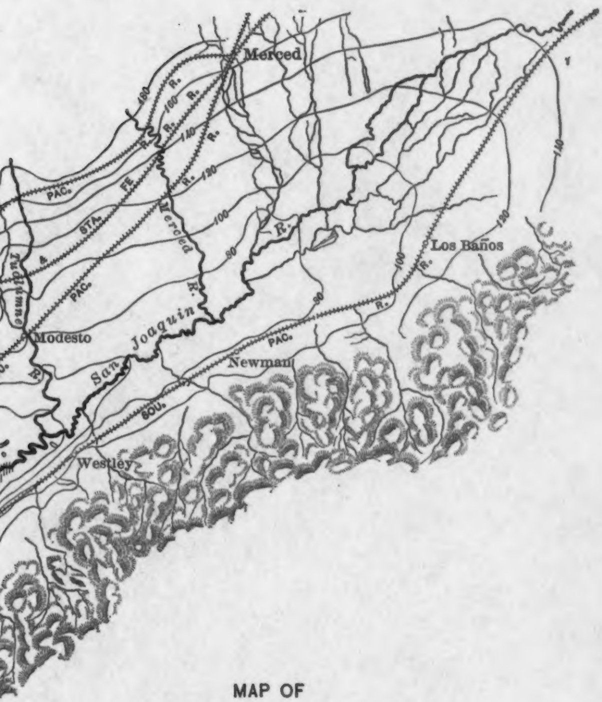
PLATE XLIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1084.
CLAPP, MURPHY AND MARTIN ON
FLOOD OF MARCH, 1907, IN CALIFORNIA RIVERS.



MAP OF
THE LOWER PORTION OF THE
SACRAMENTO AND SAN JOAQUIN VALLEYS
STATE OF CALIFORNIA

SHOWING

APPROXIMATE CONTOURS REFERRED TO MEAN SEA LEVEL,
FLOOD BASINS IN SACRAMENTO VALLEY, AND
THE MARGIN OF THE FLOOD PLANE OF
MARCH, 1907

SCALE OF MILES
5 0 5 10 15

NOTE: The margin of the flood plane shown thus, does not necessarily indicate the area actually covered by water, but it shows approximately the area which was below the water surface in the river.

the American. It has an area of 110 sq. miles, and a capacity of 571 000 acre-ft. at flood stage. It discharges into the Sacramento north of the City of Sacramento, but, owing to its great depth, it is never free from water.

The Sacramento Flood Basin is a narrow strip south of the American River, extending from the City of Sacramento to Walnut Grove. Its area and capacity are unknown. It is flooded by overflow from the Mokelumne River and the breaking of levees on the east side of the Sacramento, but not as frequently as the other basins.

What is popularly known as the San Joaquin Valley includes the real valley of the San Joaquin Basin and the valley portion of the Lake Basin. This area is one, physiographically and historically, but, for the purpose of this discussion, it has been found advantageous to divide it. What is here meant by the San Joaquin Valley, therefore, is that portion of the San Joaquin Basin below the line of the foothills. It comprises an area of about 5 890 sq. miles south of Sacramento Valley and north of Kings River and the lakes of the Lake Basin. A portion of the discharge from Kings River reaches San Joaquin River, but discussion of it is excluded from this paper because of the impossibility of estimating its amount from the data at hand. The San Joaquin Valley has no well-defined flood basins, like the Sacramento Valley; it has, however, considerable areas of marshy lands adjoining the San Joaquin River, especially along the lower course of the river, and also a large flood plane which is overflowed annually.

The Lake Basin was originally a part of the present San Joaquin Basin. Many years ago, the southern half of the Great Valley of California received its drainage from the Sierras through a series of practically parallel streams flowing in a southwest direction to the trough of the valley, where their waters were gathered into one main channel which discharged into Suisun Bay. The San Joaquin River was one of these streams. In later years, however, those streams south of the San Joaquin built up large deltas which projected into the trough of the valley. Kings River, just south of the San Joaquin, and Kern River, near the southern end of the valley, built up particularly pronounced deltas, which extended completely across the old valley trough, practically isolating that portion south of Kings River Delta. This region contains several lakes at present. Among them are Kern

and Buena Vista Lakes, south and west of Bakersfield, and Tulare Lake, about 50 miles farther northwest. Kern River flows into Kern and Buena Vista Lakes, and, during high river stages, a portion of Kings River flows into Tulare Lake. During moderately low stages, practically all the water from these streams is taken out near the head of the deltas and used for irrigation, so that only the surplus water reaches the lakes. In very wet years, a portion of the water from Buena Vista Lake may pass northward into Tulare Lake, and, under very exceptional conditions, Tulare Lake would drain into San Joaquin River. Of late years, however, there is no record of any overflow from Tulare Lake into the San Joaquin; on the other hand, the lake has been dry a portion of the time, as in 1905, although since that year it has partially filled again. Further discussion of this basin is omitted from this paper.

Along the lower courses of the Sacramento and San Joaquin Rivers a large delta has been built up. In this delta region, each of the rivers has two or more channels in certain portions of its course, especially at the higher stages. Numerous distributing and cross-sloughs extend from one channel to another, and even from one river to the other at flood stages, and many islands are thus formed. These islands are very fertile, but they are overflowed every year unless protected by levees. Many of them, more particularly those along the Sacramento River, which vary in size from 1 600 to 43 000 acres, have been reclaimed, and are now protected from overflow by levees. (See Fig. 1.)

The chief tributaries of the Sacramento River are the Pit, Feather, and American Rivers, named in order from north to south. They have their sources in the summit of the Sierras, and flow in southwesterly courses. McCloud River is the principal tributary of the Pit, and enters it from the north. Yuba and Bear Rivers are the main tributaries of the Feather, and join it from the east. The most important tributaries of the Sacramento from the west are, in order from north to south, Stony, Cache, and Puta Creeks, but the last two are lost in the flood basins, and do not really reach the main channel of the river.

San Joaquin River receives all its principal tributaries from the east. They are, in order from north to south, Mokelumne, Calaveras, Stanislaus, Tuolumne, and Merced Rivers. They rise in the Sierras

and flow westward in practically parallel courses. Cosumnes River is tributary to the Mokelumne from the northeast. (See Plate XLIV.)

Climate.

The climate of California is probably one of its most valuable assets. The principal factors affecting the climate are proximity to the Pacific Ocean, and diversified topography. The warm Japanese ocean currents, which bathe about 1 000 miles of the coast line, serve to equalize the temperature as normally affected both by seasons and latitude. The influence of the topography is such that altitude rather than latitude is the chief factor affecting temperature.

As regards precipitation, the year is divided into two well-defined seasons, the "rainy season" from November to March, and the "dry season" from April to October. The rainy season is usually marked by a series of storms, of greater or less severity, which form in the Pacific Ocean and move eastward to the coast, depositing their moisture before crossing the Sierra Nevada. The centers of the most severe storms generally strike the coast in the State of Washington and then move southward through Oregon into California between the mountain ranges. These storms almost invariably make their appearance in late winter or early spring, being, as a rule, most severe about the time of the vernal equinox. At this season the precipitation is quite general throughout the State, increasing with altitude and also with latitude.

Flood Conditions and Causes.

During the winter and early spring of each year, toward the end of the rainy season, the various streams of the Sacramento and San Joaquin Basins generally reach their highest stages. The most serious flood conditions invariably exist on the lower courses of the trunk streams, the Sacramento and the San Joaquin. On the Sacramento River, in particular, serious damage is inflicted on crops and transportation interests almost every year. Of course, the destructiveness of any flood is measured largely by its height and duration. In these basins the maximum height, and generally the greatest duration, of floods on the primary streams, result from the simultaneous flooding of all the secondary and tertiary streams, a condition which obtains when there is a period of long-sustained precipitation throughout the entire water-shed, accompanied by high temperature and rapid melting of snow on the higher elevations. It was such a condition that brought about the flood of March, 1907.

Other conditions that contribute more or less to all floods in this area are the following:

1.—The steep, barren, and impervious slopes of the mountains and foot-hills, which result in streams of heavy grades and the rapid delivery of water to the valleys;

2.—The broad, flat valleys, with light grades and sluggish streams;

3.—The limited channel capacity. It is said that some of the trunk channels are not large enough to carry even one-third of the flood flow. Particularly is this true of the Sacramento River. Here the surplus water overflows into the flood basins, the result being either to increase or diminish the stage of the lower course of the river, depending on the volume of water in the flood basins at the beginning of the flood period and the duration of the period.

4.—The common outlet of the two river systems, with large tributaries of each system discharging into trunk streams near this outlet;

5.—The constriction of the flood area in the delta of the two rivers through the reclamation of large areas of overflow land by levees;

6.—The deposition of the débris resulting from hydraulic mining in several tributaries of the Sacramento River, the result of which has been the filling of channels and the reduction of gradients, thereby raising the flood plane several feet;

7.—The tidal and wind action in the delta of the two rivers.

Precipitation.

In the Sacramento Valley, the mean annual precipitation varies from 15 in. in the southern to 20 in. in the northern part, while, in the tributary foot-hill and mountain areas, it varies from 20 to 60 in., with an occasional maximum of 100 in. In the San Joaquin Valley, the mean annual precipitation varies from 10 in. in the southern to 15 in. in the northern part, and in the foot-hill and mountain areas it varies from 15 to 40 in. In the Sierras, the greater part of the precipitation is normally in the form of snow, and the magnitude of floods depends largely on its rate of melting. A heavy, warm rain on a deep, freshly fallen snow produces a maximum run-off.

In January and February, 1907, there were two periods of heavy and long-sustained precipitation, one from January 2d to 17th, and the other from January 24th to February 4th. The precipitation was unusually heavy over the Sacramento Basin, diminishing gradually

toward the south. The precipitation during the first of these periods produced the ordinary winter stages on the tributaries of the Sacramento River; that during the second period produced flood stages on the tributaries of both the Sacramento and San Joaquin Rivers and high stages in the Lower San Joaquin. American and Bear Rivers reached stages almost as high as in the great flood of the following month. Yuba River was higher than at any time previously recorded.

In March there were two precipitation periods, one from the 2d to the 11th, in which the amount of rainfall was moderate, and the other from the 16th to the 25th, in which it was extraordinarily heavy. The precipitation of the latest period was accompanied by unusually high mean temperatures, especially in the higher altitudes, from the Feather River south to the Tuolumne, causing very rapid melting of snow and exceedingly large run-off. The average from 24 fairly representative meteorologic stations throughout the basin shows that the mean temperature for March 17th to 20th was about 5° above the mean for the month, with low daily maxima resulting from cloudiness and rain, and high daily minima due to the liberation of heat by the storm. The average greatest daily range in this period was only 16 degrees. These facts indicate that probably all stations with a monthly mean temperature as high as 25° had scarcely any freezing conditions from March 16th to 20th, when the precipitation was heaviest. Further, they show that, out of 113 stations located at various elevations throughout the Sacramento and San Joaquin drainage basins, at 105 of them all the precipitation from March 17th to 20th was probably in the form of rain or of snow in a melting condition.

Table 1 shows the monthly precipitation from January to March, 1907, the daily precipitation for the three days, March 17th, 18th, and 19th, when it was greatest, and the precipitation for the ten days, March 17th to 26th, for 120 places in the Sacramento and San Joaquin Basins, varying in altitude from 20 to 7 017 ft., arranged according to basins of tributary streams. Where possible, the monthly precipitation for the period of January to March, 1904, is also given, for comparison with the great flood of that year.

Table 2 shows the average precipitation and average mean temperatures for March at 113 stations arranged according to stream basins and in order of altitude.

Table 3 shows the results at 24 stations ranging in altitude from 60 to 5 270 ft., the data having been taken from the Climatological Report of the United States Weather Bureau for March, 1907. These particular stations were selected because they are the only ones in the basin for which daily temperatures have been published. They are fairly well distributed, both as regards area and altitude, and are probably as representative as any that could have been chosen. This table also shows the extraordinary intensity of precipitation from March 17th to 26th by percentages with reference to the total for the month, and also the normal for the month, covering a period of 21 years on an average.

These tables show conclusively that the precipitation from March 17th to 26th, and particularly on March 17th, 18th, and 19th, was phenomenally heavy for this section of the country. This large precipitation is rather evenly distributed throughout all the river basins, but there is a very noticeable and quite rapid, though comparatively regular, increase with the altitude. During the month, sixteen stations, with elevations of more than 3 500 ft., had more than 30 in. in depth of precipitation; about forty stations, with elevations of more than 1 500 ft., had more than 20 in.; and fully one-third of the total precipitation for the month fell on March 17th, 18th, and 19th. On one of these three days, seventeen stations, with an altitude of more than 2 000 ft., had precipitations of from 5 to 8 in. in 24 hours. It is noteworthy that the range of temperature with altitude was quite regular, and that there were no very low temperatures even at very high elevations. It is highly probable that at elevations of 5 000 ft. a large part of the precipitation occurred as rain or as snow which melted rapidly. Indeed, at Inskip, in the Feather Basin, with an elevation of 4 850 ft., a 24-hour rainfall of 8 in. was reported. Taking a record of 21 years on an average throughout the basin, it is seen that about 88% of the normal precipitation for March occurred on March 17th, 18th, and 19th, 1907, or, counting 20 days as normally rainy in this month, the intensity of this 3-day period was about 600% of the normal intensity for the month. During these 3 days the average precipitation at the sixteen stations, principally in the Feather and Yuba Basins, having more than 30 in. during the month, was 145% of the normal for the month, or at an average intensity of 1 000% of the normal.

TABLE 1.—PRECIPITATION.

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	
	Sacramento Drainage Basin.											
1	Delta	1 138	1904	8.96	21.19	23.93	49.08					
			1907	12.26	9.21	24.45	45.92	1.10	3.50	3.50	18.10	47.3°
2	Redding	565	1904	2.34	14.10	15.89	32.33					
			1907	8.57	9.09	7.28	24.94	1.36	0.88	0.11	4.57	49.0°
3	Dunsmuir	2 285	1904	5.03	24.00	22.90	51.93					
			1907	20.53	8.27	18.64	47.44	2.00	2.87	2.72	13.61	46.9°
4	Sisson	3 555	1904	3.26	10.91	15.90	30.07					
			1907	9.48	2.84	13.16	25.48	0.00	3.55	1.27	11.27	37.6°
5	Nimshew	2 000	1904	10.82	8.72	13.27	32.81					
			1907	17.64	13.12	27.69	58.45	5.54	4.00	3.02	18.36	42.4°
6	Sacramento	71	1904	0.45	5.26	5.43	11.14					
			1907	4.63	2.37	7.28	14.28	0.42	1.74	0.56	4.75	50.9°
7	Fruto	624	1904	0.75	6.13	7.07	13.95					
			1907	6.43	1.95	4.67	13.05	0.45	0.40	0.30	2.90	50.1°
8	Shasta	1 148	1904	2.79	24.86	16.37	44.02					
			1907	13.65	7.89	14.47	36.10	1.54	2.25	1.03	10.98	48.6°
9	Corning	277	1904	0.60	4.95	7.30	12.85					
			1907	3.60	2.60	5.05	11.25	0.00	0.95	0.28	2.68	49.1°
10	Red Bluff	307	1904	1.44	6.63	8.33	16.40					
			1907	6.10	3.13	5.92	15.15	0.62	0.28	0.00	2.93	48.4°
11	Tehama	220	1904	1.01	4.67	7.19	12.87					
			1907	4.75	2.96	5.38	13.09	0.86	0.28	0.30	2.79	50.8°
12	Chico	189	1904	0.80	5.64	9.33	15.77					
			1907	6.28	2.09	8.08	18.67	1.58	0.73	0.36	4.79	49.2°
13	Durham	160	1904	1.70	5.75	10.32	17.77					
			1907	6.45	2.09	8.39	16.93	1.61	0.87	0.45	4.49	50.4°
14	Willows	136	1904	0.45	3.44	7.61	11.50					
			1907	4.84	1.02	3.63	9.48	0.70	0.13	0.05	1.98	49.1°
15	Colusa	60	1904	0.66	3.13	5.67	9.46					
			1907	5.63	0.75	3.80	10.18	0.58	0.22	0.00	2.21	49.9°
16	Suisun	20	1904	1.12	6.50	7.52	15.14					
			1907	8.89	3.59	7.57	20.05	1.61	0.08	1.55	5.95	52.0°
17	Dunnigan	65	1904	0.66	5.33	8.87	14.86					
			1907	7.63	1.63	6.98	16.24	1.10	0.04	1.40	3.91	52.9°
18	West Branch	3 150	1907	......	17.96	39.69	......	....	....	....	......	.....

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	
	McCloud River Drainage Basin.											
19	Johns Camp	......	1904	4.50	19.73	27.26	51.49					
	Pit River Drainage Basin.											
20	Cedarville	4 675	1904	1.12	4.87	4.61	10.60					
			1907	1.99	3.70	3.31	9.00	0.67	0.40	0.06	1.59	34.4°
21	Alturas	4 460	1907	1.35	2.87	4.13	8.35	0.55	0.75	0.14	2.45	35.6°
	Feather River Drainage Basin.											
22	Magalia	2 321	1904	3.43	23.39	30.13	56.95					
			1907	23.57	10.71	37.75	72.03	7.65	6.66	2.79	24.42	41.9°
23	Oroville	250	1904	1.60	7.99	10.86	20.45					
			1907	6.71	3.59	10.90	21.20	1.10	1.44	0.52	5.57	51.0°
24	Butte Valley	4 020	1904	4.20	22.90	22.10	49.20					
			1907	11.96	6.78	26.76	45.50	....	....	....	......	......
25	Greenville	3 600	1904	2.39	18.81	15.53	36.73					
			1907	9.57	4.48	24.51	38.56	4.25	6.17	2.91	19.89	37.8°
26	Meadow Valley	4 730	1904	4.13	29.10	29.90	63.13					
27	Quincy	3 400	1904	2.46	22.10	10.83	35.39					
			1907	11.89	4.96	30.15	47.00	5.30	6.50	4.40	25.55	35.8°
28	Inskip	4 850	1907	......	......	45.30	......	....	....	....	......	......
29	Biggs	98	1904	1.09	4.98	8.35	14.42					
			1907	4.55	1.85	6.57	12.97	0.20	0.00	0.00	3.65	52.0°
30	Brush Creek	2 140	1904	4.81	23.11	25.01	52.93					
			1907	16.21	11.49	33.02	60.72	5.70	5.40	3.40	23.96	42.8°
31	Marysville	67	1904	1.19	5.18	7.77	14.14					
			1907	4.52	4.30	10.59	19.41	0.30	1.30	2.00	6.44	53.8°
32	Palermo	213	1904	1.48	7.22	9.35	18.05					
			1907	5.36	3.34	8.80	17.50	0.97	0.86	0.25	4.37	50.8°
33	Sterling City	3 525	1904	3.96	26.51	25.22	55.69					
			1907	24.63	17.54	43.38	85.55	6.66	7.90	6.16	32.86	37.0°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	
	Yuba River Drainage Basin.											
34	Colgate	650	1904	3.79	9.92	8.19	21.90					
			1907	7.86	10.28	19.31	37.45	....	....	....	......	......
35	Dobbins	1 650	1904	3.79	14.04	13.65	31.48					
			1907	10.54	8.98	19.43	38.95	2.50	2.33	2.60	13.04	......
36	Nevada City	2 580	1904	2.76	19.17	18.64	40.57					
			1907	10.21	8.22	24.62	43.05	2.47	3.34	3.63	17.76	41.6°
37	No. Bloomfield	3 200	1904	3.85	16.44	21.89	42.18					
			1907	10.25	9.23	28.64	48.12	3.07	4.51	3.97	21.10	39.8°
38	Cisco	5 939	1904	5.20	30.80	26.87	62.87					
			1907	14.70	6.25	24.20	45.15	1.00	1.00	3.60	14.10	34.4°
39	Summit	7 017	1904	4.20	30.40	21.30	55.90					
			1907	13.50	4.38	27.36	45.24	1.42	2.42	2.32	16.06	28.8°
40	La Porte	5 000	1904	4.48	30.35	31.66	66.49					
			1907	17.75	16.40	42.62	76.27	6.58	6.19	5.42	33.12	32.2°
41	Comptonville	3 400	1907	......	16.78	36.12	......	....	....	....	......	......
42	Woodleaf	3 250	1907	18.75	12.87	37.38	69.00	....	....	....	......	......
43	Deer Creek	......	1907	......	......	36.93	......	....	....	....	......	......
44	Head Dam	......	1907	......	......	26.78	......	....	....	....	......	......
45	Fordyce Dam	6 500	1907	12.11	11.26	29.01	52.38	....	....	....	......	......
46	Bowman's Dam	5 500	1904	5.37	45.61	39.51	90.49					
			1907	13.82	12.68	31.46	57.96	....	....	....	......	......
	Bear River Drainage Basin.											
47	Bear Valley	4 600	1904	4.46	34.26	27.99	66.71					
			1907	14.59	11.10	35.50	61.19	....	....	....	......	......
48	Wheatland	84	1904	1.09	6.14	7.22	14.45					
			1907	4.67	3.06	9.64	17.37	1.18	1.23	0.81	6.19	50.6°
49	Grass Valley	2 090	1907	11.22	11.79	26.15	49.16	....	....	....	......	......
50	Gold Run	3 222	1907	10.47	9.61	21.61	41.69	....	....	....	......	39.4°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean Temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	
	American River Drainage Basin.											
51	Colfax	2 421	1904	3.50	20.10	20.46	44.06					
			1907	9.45	9.75	19.46	38.66	2.40	1.55	2.85	12.36	48.2°
52	Emigrant Gap	5 230	1904	3.75	25.10	31.22	60.07					
			1907	14.35	14.45	30.20	59.00	2.00	2.50	4.50	19.45	30.0°
53	Georgetown	2 650	1904	4.79	26.02	21.17	51.96					
			1907	8.96	13.50	29.07	51.53	3.58	1.08	4.90	19.47	42.4°
54	Placerville	2 109	1904	2.96	15.59	13.48	32.03					
			1907	8.13	8.15	20.54	36.82	2.03	1.06	4.28	14.62	47.0°
55	Rocklin	249	1904	1.29	7.94	2.18	16.41					
			1907	5.51	5.71	12.46	23.68	1.70	0.15	2.20	8.65	51.2°
56	Represa	305	1904	1.15	8.33	8.55	18.03					
			1907	6.31	5.31	12.39	24.01	....	....	....	.....	.....
57	Auburn	1 360	1904	2.73	13.34	11.83	27.90					
			1907	8.35	9.70	16.66	34.71	2.08	0.39	3.11	11.23	47.6°
58	Blue Canyon	4 695	1904	4.81	30.61	26.14	61.56					
			1907	13.18	17.95	35.11	66.24	4.18	4.35	6.45	27.33	36.6°
59	Iowa Hill	2 825	1904	4.58	20.20	16.97	41.75					
			1907	11.52	10.13	24.36	46.01	2.42	2.88	3.21	16.35	42.6°
60	New Castle	970	1904	1.93	10.79	11.61	24.33					
			1907	7.09	6.72	14.10	27.91	1.35	1.18	1.78	8.49	50.0°
61	Folsom	252	1904	1.12	7.19	7.70	16.01					
			1907	5.25	5.65	11.06	21.96	1.42	0.10	2.08	7.33	51.0°
62	Pilot Creek	4 000	1904	5.48	29.88	25.45	60.81					
			1907	14.40	11.79	32.88	59.07	....	....	....	......	.....
63	Towle	3 704	1904	3.84	25.50	23.29	52.63					
			1907	9.45	12.24	24.05	45.74	2.40	2.43	2.69	15.83	37.7°
	Stony Creek Drainage Basin.											
64	Fouts Springs	1 650	1904	2.34	9.44	12.73	24.51					
			1907	14.85	4.60	15.63	35.08	....	....	....	......	.....
65	Julian	750	1904	0.75	4.79	6.22	11.76					
66	Orland	254	1904	......	4.06	6.36	......					
			1907	......	......	3.97	......	0.57	0.15	0.12	2.11	48.8°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total. 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th-26th.	
	Cache Creek Drainage Basin.											
67	Bartlett Springs..........	2 375	1904	2.48	19.96	16.75	39.19					
68	Kono Toyee..............	1 350	1904	1.64	8.78	7.62	18.04					
69	Lake Port................	1 325	1904	1.65	13.37	12.72	27.74					
70	Upper Lake	1 350	1904	1.62	11.19	10.14	22.95					
			1907	5.30	4.60	10.63	20.53	2.40	1.73	0.70	7.98	47.6°
71	Guinda	350	1904	0.75	6.80	7.55	15.10					
			1907	9.30	1.30	8.84	19.44	1.30	1.00	1.70	7.20	47.9°
72	Woodland.................	63	1904	0.69	4.60	7.15	12.44					
			1907	4.45	3.24	5.90	13.59	....	....	....	4.10	50.5°
	Puta Creek Drainage Basin.											
73	Middletown................	1 800	1904	2.52	16.99	27.57	47.08					
74	Davisville	51	1904	0.53	5.05	7.57	13.15					
			1907	4.81	2.28	6.69	13.78	1.25	0.07	2.00	5.24	56.1°
75	North Lake Port..........	1 450	1907	5.45	4.30	12.35	22.10	....	....	....	......	......
76	Calistoga..................	363	1904	2.65	16.08	16.10	34.83					
			1907	10.89	7.95	19.50	38.34	0.00	5.60	3.85	16.60	52.6°
77	Helen Mine................	2 750	1904	4.52	34.22	31.48	70.22					
			1907	27.21	11.66	36.73	75.60	7.40	6.64	5.10	28.90	43.4°
78	Vacaville..................	175	1904	1.67	8.61	11.73						
			1907	6.54	3.08	8.48	18.10	0.13	2.02	0.29	4.81	49.7°
79	Mt. St. Helena	2 300	1904	3.37	28.34	26.14	57.85					
			1907	19.95	12.18	24.20	56.33	....	....	....	......	......

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th-26th.	
	San Joaquin Drainage Basin.											
80	Farmington	111	1904	0.54	4.71	4.10	9.35					
			1907	4.70	2.65	5.47	12.82	....	....	....	......	52.9°
81	Fresno	293	1904	0.57	2.49	2.75	5.81					
			1907	3.35	0.94	1.74	6.03	0.12	0.00	0.32	0.56	52.8°
82	Le Grand	255	1904	1.15	2.60	2.90	6.65					
			1907	4.40	0.77	5.56	10.73	0.00	0.45	0.00	2.96	47.3°
83	Las Banas	121	1904	0.25	1.23	1.28	2.76					
			1907	3.17	1.17	4.39	8.73	0.00	0.45	0.67	2.78	53.0°
84	Mendota	177	1904	0.20	1.70	1.26	3.16					
			1907	2.83	1.31	1.79	5.93	0.00	0.03	0.00	0.64	55.5°
85	Merced	173	1904	0.55	2.30	2.34	5.19					
			1907	4.25	3.16	3.68	11.00	0.00	0.00	0.22	2.73	51.7°
86	Newman	91	1904	0.23	1.51	2.33	4.07					
			1907	3.35	1.49	3.82	8.66	0.15	0.00	0.67	2.42	51.8°
87	Stockton	23	1904	0.54	4.09	3.67	8.30					
			1907	3.94	2.52	6.03	12.49	0.53	0.06	1.22	4.07	51.2°
88	Storey	296	1904	0.69	2.69	2.47	5.85					
			1907	2.70	0.48	1.35	4.53	0.00	0.00	0.01	0.47	49.8°
89	Tracy	64	1904	0.46	2.10	1.93	4.49					
			1907	3.22	1.70	5.04	9.96	0.00	0.00	0.70	2.75	48.0°
90	Westley	90	1904	0.41	1.53	3.07	5.01					
			1907	5.18	1.39	3.55	10.12	0.00	0.18	0.48	2.31	55.2°
91	No. Fork	3000	1904	......	......	10.73	......					
			1907	9.19	4.42	14.30	27.91	....	....	....	......	.....
92	Pollasky	345	1907	4.20	0.86	4.24	9.30	....	....	....	......	.....
93	Lathrop	25	1907	3.58	1.54	4.64	9.76	....	....	....	......	41.5°
94	Antioch	46	1904	0.42	2.65	4.65	7.72					
			1907	3.23	1.80	6.43	11.46	0.65	0.03	0.83	4.38	54.8°

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar. 18th.	Mar. 19th.	Mar. 17th–26th.	
	Cosumnes Drainage Basin.											
95	Shingle Springs...........	1 427	1904	2.80	15.91	12.58	31.29					
			1907	5.05	8.50	16.84	30.39	....	....	....	......	......
	Mokelumne River Drainage Basin.											
96	Mill Creek.................	3 500	1907	10.20	8.00	24.45	42.65	2.06	2.36	4.04	15.96	41.5°
97	Kennedy Mine.............	1 500	1904	2.08	13.79	9.22	25.09					
			1907	6.25	6.25	13.85	26.35	....	....	....	......	......
98	Electra.....................	725	1904	2.61	13.92	9.50	26.03					
			1907	7.47	5.15	18.01	30.63	0.26	4.55	0.47	10.04	53.2°
99	Mokelumne Hill...........	1 560	1904	2.44	13.35	9.52	25.31					
			1907	7.61	6.29	15.66	29.56	....	....	....	......	48.0°
100	West Point....	2 326	1904	4.62	16.36	13.49	34.47					
			1907	9.38	6.66	19.76	35.80	....	....	....	......	......
101	Galt........................	49	1904	0.60	6.24	5.27	12.11					
			1907	4.00	3.29	7.59	15.38	1.00	0.04	1.47	5.92	51.3°
102	Ione........................	287	1904	0.90	7.05	5.00	12.95					
			1907	4.87	3.95	10.39	19.21	1.10	0.11	2.40	6.27	44.9°
103	Lodi........................	35	1904	0.72	5.77	4.85	11.34					
			1907	3.94	2.82	6.76	13.52	0.42	0.21	1.18	4.26	50.7°
	Calaveras Drainage Basin.											
104	Milton......................	660	1904	0.93	6.78	5.30	13.01					
			1907	4.76	2.53	9.27	16.56	0.34	0.35	1.53	4.90	51.2°
105	Valley Springs............	673	1904	1.42	10.56	7.81	19.79					
			1907	5.51	4.31	11.12	20.94	0.92	0.25	2.62	6.66	54.2°
106	Jenny Lind................	300	1907	......	2.61	9.38	......	....	....	....	......	......

TABLE 1.—(*Continued.*)

Rainfall Stations.				Precipitation, in Inches.								Mean temperature for March.
No.	Name.	Elevation.	Date.	Jan.	Feb.	Mar.	Total, 3 months.	Mar. 17th.	Mar 18th.	Mar. 19th.	Mar. 17th–26th.	
	Stanislaus River Drainage Basin.											
107	Oakdale	156	1904	0.70	5.00	3.44	9.14					
			1907	3.72	2.36	6.37	12.45	0.59	0.00	1.33	3.36	49.1°
108	Melones	760	1907	5.03	6.49	17.48	29.00	....	....	....	......	......
109	Relief Creek	7 500	1907	11.38	2.63	29.43	43.44	....	....	....	......	......
	Tuolumne River Drainage Basin.											
110	Jamestown	1 471	1904	1.96	12.96	8.18	23.10					
			1907	7.82	5.59	17.27	30.58	1.04	1.43	1.83	10.68	49.0°
111	Tuol. Camp	3 100	1907	9.89	4.92	20.79	35.60	....	....	....	......	......
112	Crocker's (Sequoia P. O.)	4 452	1904	1.87	17.10	19.56	38.53					
			1907	13.49	5.82	27.41	46.72	...	....	....	......	......
113	Groveland	3 100	1907	9.66	3.70	15.95	29.31	....	....	....	......	......
114	Jacksonville	850	1907	5.76	4.54	13.38	23.68	....	....	....	......	......
115	Modesto	90	1904	0.33	1.67	2.15	4.15					
			1907	4.11	3.00	4.70	11.81	0.34	1.05	0.76	3.64	57.3°
116	Sonora	1 900	1904	1.79	13.82	8.63	24.24					
			1907	7.38	5.40	19.09	31.87	1.41	1.46	1.93	12.17	47.4°
	Merced River Drainage Basin.											
117	Summerdale	5 270	1904	2.60	14.96	17.09	34.66					
			1907	14.95	6.81	27.06	48.82	2.04	0.71	1.90	13.90	35.2°
118	Yosemite	3 945	1904	2.99	13.95	12.53	29.47					
			1907	11.96	3.72	20.98	36.66	2.61	2.02	1.85	13.15	38.4°
119	Merced Falls	375	1907	4.04	2.08	5.85	11.97	....	....	....	......	......
120	Elmwood	126	1904	0.57	2.19	2.17	4.93					
			1907	3.60	0.60	3.26	7.46	0.00	0.00	0.22	1.64	52.6°

TABLE 2.—RAINFALL STATIONS, ACCORDING TO ELEVATION, FOR MARCH, 1907.

ELEVATION, IN FEET.	0–500 46 STATIONS.		500–1 000 10 STATIONS.		1 000–2 000 12 STATIONS.		2 000–3 000 16 STATIONS.		3 000–4 000 15 STATIONS.		4 000–5 000 7 STATIONS.		5 000– 6 STATIONS.
Basin.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.	Mean temperature.	Total precipitation.
Sacramento	6.20	50.3°	5.98	49.6°	19.46	48.0°	23.16	44.6°	26.42	37.6°	.....	.....	.....
Feather	9.22	51.9°	.....	.....	.....	.....	35.38	42.4°	32.68	36.9°	36.03	.....	.....
Yuba	.....	.....	19.31	.....	19.43	.....	25.70	41.6°	34.77	39.8°	42.62	32.2°	28.01
Bear	9.64	50.6°	.....	.....	.....	.....	26.15	.....	21.61	39.4°	35.50	.....	.....
American	11.97	51.1°	14.10	50.0°	16.66	47.6°	23.36	45.0°	28.46	37.7°	35.11	36.6°	30.20
Cosumnes	.....	.....	.....	.....	16.84	.....	.....	.....	.....	.....	.....	.....	.....
Mokelumne	8.25	49.0°	18.01	53.2°	14.76	48.0°	19.76	.....	24.45	41.5°	.....	.....	.....
Calaveras	9.38	.....	10.20	52.7°	.....	.....	.....	.....	.....	.....	.....	.....	.....
Stanislaus	6.37	49.1°	17.48	.....	.....	.....	.....	.....	.....	.....	.....	.....	29.43
Tuolumne	4.70	57.3°	13.38	.....	18.18	48.2°	20.79	.....	15.95	.....	27.41	.....	.....
Merced	4.56	52.6°	.....	.....	.....	.....	.....	.....	20.98	38.4°	27.06	35.2°	.....
San Joaquin	4.12	51.2°	.....	.....	.....	.....	14.30	.....	.....	.....	.....	.....	.....
Stony Cr.	3.97	48.8°	6.22	.....	15.63	.....	.....	.....	.....	.....	.....	.....	.....
Cache Cr.	7.37	49.2°	.....	.....	10.63	47.6°	.....	.....	.....	.....	.....	.....	.....
Puta Cr.	11.56	52.8°	.....	.....	12.35	.....	30.46	43.4°	.....	.....	.....	.....	.....
Average of all Stations	6.65	51.0°	12.09	51.6°	16.36	48.2°	25.24	43.9°	28.66	38.3°	34.25	34.7°	28.61

A comparison of the precipitation from January to March, 1907, with that for the same period and stations in 1904, shows that, for the average of all stations in the water-shed, the precipitation was greater in 1907 than in 1904, and that the difference increases from the north toward the south, but the percentage in favor of the former is quite small. An examination of Table 1 shows that the precipitation for January, 1904, was quite light compared with that of January, 1907, while the precipitation for February, 1904, was much heavier than for February, 1907. The comparison for March, however, is of most importance, as regards the floods of 1904 and 1907. Such a comparison is made in Table 4, where it is seen that, with the exception of the Sacramento River Basin, the precipitation throughout the water-shed was much greater in 1907. In the basins of the tributaries of the Sacramento River from the east, the rainfall in March, 1907, was from 20 to 41% greater than in March, 1904, while for basins on the west it is only from 2 to 3% greater. For the San Joaquin River and its tributaries the percentage is much greater, ranging from about 50 to 80 per cent. The distribution of the precipitation during the

TABLE 3.—RESULTS AT TWENTY-FOUR STATIONS, RANGING IN ALTITUDE FROM 60 TO 5 270 FEET.

Station.	Drainage.	Length of record, in years.	Precipitation in March, 1907.					Temperature.		
			Month.		March 17th-26th.	March 17th-19th.		Month.	March 17th-20th.	
			Total, in inches.	Percentage above normal.	Percentage of total.	Percentage of total.	Percentage of normal monthly.	Mean.	Mean.	Maximum daily range.
Alturas	Pit	3	4.13	....	59	35	....	35.6°	43.0°	13°
Auburn	American	36	16.66	224	67	33	107	47.6°	45.8°	19°
Cedarville	Pit	13	3.31	124	48	16	37	34.4°	42.0°	19°
Chico	Sacramento.	37	8.03	197	60	32	95	49.2°	53.1°	16°
Colusa	Sacramento.	24	3.80	28	58	21	27	49.9°	57.2°	18°
Fresno	San Joaquin	20	1.74	32	32	25	33	52.8°	58.1°	18°
Greenville	Feather	13	24.51	371	81	54	254	37.8°	40.5°	14°
Merced	San Joaquin	33	3.68	156	74	6	15	51.7°	52.5°	25°
Milton	Calaveras	17	9.27	133	53	24	56	51.2°	58.3°	13°
Nevada City	Yuba	15	24.62	193	69	38	111	41.6°	48.2°	19°
No. Blmfld	Yuba	10	28.64	254	74	44	156	39.8°	45.5°	23°
Palermo	Feather	16	8.80	247	50	23	80	50.8°	55.5°	17°
Quincy	Feather	12	30.15	353	85	54	244	35.8°	41.4°	16°
Red Bluff	Sacramento.	30	5.92	81	50	15	27	48.4°	52.2°	10°
Redding	Sacramento.	32	7.28	53	63	32	49	49.0°	52.9°	10°
Sacramento	Sacramento.	30	7.28	148	65	37	92	50.9°	56.6°	12°
Shasta	Sacramento.	11	14.47	167	76	33	88	48.6°	47.4°	19°
Stockton	San Joaquin	36	6.03	164	67	30	79	51.2°	57.4°	18°
Summerdale	Merced	11	27.06	194	51	17	50	35.2°	39.4°	9°
Upper Lake	Cache	22	10.63	238	75	45	152	49.6°	50.8°	16°
Wheatland	Bear	20	9.64	247	64	33	109	50.6°	56.7°	17°
Willows	Sacramento.	28	3.63	119	55	24	53	49.1°	54.7°	17°
Yosemite	Merced	3	20.98	....	63	31	....	38.4°	39.5°	17°
Georgetown	American	34	29.07	209	67	33	102	42.4°	....	......
Average of above 24 Stations		21	12.43	179	63	31	87	45.5°	50.7°	16°
Average of 71 Stations in Basin		....	12.96	185	....	....	....	46.5°	... .	......

NOTE:—These selected stations are fairly representative as regards both temperature and precipitation. It is observed that the average of the mean temperature, March 17th-20th, is 5.2° above that for the month.

month, however, is of most vital significance. For March, 1904, the precipitation is distributed quite evenly throughout the entire month, though the intensity is noticeably greater during the equinoctial week. For March, 1907, however, not only is the total precipitation for the month considerably greater than in 1904, but its periodic occurrence in a series of storms is more pronounced. During the 10-day period centering about the equinox, the intensity was so great that about 70% of the total precipitation for the month occurred in this time, while more than 30% of it was recorded on March 17th, 18th, and 19th.

TABLE 4.—COMPARISON OF PRECIPITATION IN SACRAMENTO AND SAN JOAQUIN BASINS, FOR MARCH, 1904, AND MARCH, 1907.

River basin.	Number of precipitation stations.	Precipitation.		Percentage of Difference.	
		1904.	1907.	1904.	1907.
				12	....
Sacramento	17	11.34	10.13	....	
Feather	10	16.52	23.24	....	41
Yuba	8	22.71	27.20	....	20
Bear	2	17.60	22.57	....	28
American	11	20.46	25.67	....	25
Stony	2	9.54	9.80	....	3
Cache	3	8.28	8.46	....	2
Puta	5	18.60	19.12	....	3
San Joaquin	13	3.34	4.86	....	45
Cosumnes	1	12.58	16.84	....	34
Mokelumne	7	6.56	10.20	....	56
Calaveras	2	8.12	13.15	... .	62
Stanislaus	1	3.44	6.37	....	85
Tuolumne	4	9.63	17.12	....	78
Merced	3	10.60	17.10	....	61

FLOOD FLOW OF STREAMS.

On the following pages is recorded the daily flow of the various streams in the Sacramento and San Joaquin Basins for the 11 days, March 16th-26th, also the mean daily flow for the 4-day period, March 18th-21st, at all places where gauging stations were maintained. The figures given herein are not the final figures as they may appear in the Annual Report of the United States Geological Survey, but they will not differ materially from them. In all cases it is believed that the estimates are quite conservative, and rather inclined to be too low than too high.

Pit River.—This river drains a long, comparatively narrow, and high mountainous area in the northeastern part of the Sacramento Basin. In this area are several large reservoir sites. Those surveyed, to June, 1905, have a capacity of 6 000 000 acre-ft., but Big Valley Reservoir, above Bieber, with a capacity of 3 196 000 acre-ft., is in all probability the only one that could be utilized for flood control. The precipitation in this basin above Bieber was comparatively light, and occurred mainly as snow, so that the run-off per square mile was small.

The gauging station is about 12 miles below Bieber and about 70 miles in a direct line above the mouth of the McCloud River. The area above this gauging station is 2 950 sq. miles. Table 5 contains data on the flood flow at this station during the flood of 1907.

TABLE 5.—FLOW OF PIT RIVER, NEAR BIEBER.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.5	2 770
" 17th	12.8	17 400
" 18th	15.5	25 000
" 19th	16.4	27 500
" 20th	15.5*	25 000
" 21st	14.0*	20 800
" 22d	11.5*	13 800
" 23d	9.7	8 810
" 24th	8.5	6 110
" 25th	7.4	4 160
" 26th	7.3*	4 000

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	4 180	232 000
March	6 940	427 000
March 18th-21st	24 600	195 000

* Estimated.

The mean rate of flow for the 24 hours when it was greatest was 9.3 cu. ft. per sec. per sq. mile, and the mean rate for the 4 consecutive days, March 18th-21st, was 8.3 cu. ft. per sec. per sq. mile. This small run-off was due to light precipitation and to the slow melting of the snow.

McCloud River.—McCloud River, the principal tributary of the Pit River, drains a long, narrow, mountainous, timbered strip of about 676 sq. miles on the north side of the Pit River Basin, including the southern and eastern slopes of Mount Shasta. Its low-water flow is remarkably large, never having been less than 1 200 cu. ft. per sec. at the gauging station in 4 years.

The gauging station is 14 miles east of Baird Spur, on the Southern Pacific Railroad, at Gregory Post-Office, and the drainage area above it is 608 sq. miles. Table 6 contains data on the flow at this station during the flood of 1907.

The mean rate of flow at this station for the 24 hours when it was greatest was 50.0 cu. ft. per sec. per sq. mile, and the mean for the 4 consecutive days, March 18th-21st, was 35.5 cu. ft. per sec. per sq. mile.

Upper Sacramento River.—The gauging station on the Upper Sacramento is in the foot-hills near Iron Canyon, 4 miles above Red Bluff, at an elevation of about 310 ft. above sea level. The drainage

area above it includes 9 300 sq. miles of mountains and foot-hills. About 38% of this area is above the stations on Pit and McCloud Rivers. Table 7 contains data on the flow of the river at this place during the floods of 1907 and 1904.

TABLE 6.—FLOW OF McCLOUD RIVER, NEAR GREGORY.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	2.8	2 460
" 17th	4.0	4 210
" 18th	9.4	19 200
" 19th	12.0	30 400
" 20th	10.65	24 200
" 21st	7.5	12 700
" 22d	5.9	8 360
" 23d	5.5	7 400
" 24th	4.9	6 060
" 25th	4.55	5 300
" 26th	3.95	4 120

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	5 490	305 000
March	5 990	368 000
March 18th–21st	21 600	171 000

TABLE 7.—FLOW OF UPPER SACRAMENTO RIVER, NEAR RED BLUFF.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.	Date, 1904.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.6	23 600	March 7th	16.30	77 200
" 17th	10.0	39 100	" 8th	24.40†	147 130
" 18th	21.4	118 000	" 9th	18.95	97 200
" 19th	26.05	164 000	" 10th	17.90	88 940
" 20th	28.7*	192 000	" 11th	15.80	73 700
" 21st	22.85	132 000	" 12th	14.70	66 220
" 22d	18.4	92 900	" 13th	13.30	57 400
" 23d	21.65	120 000	" 14th	15.80	73 700
" 24th	16.8	80 800	" 15th	17.25	84 050
" 25th	14.3	64 000	" 16th	18.30	92 040
" 26th	13.25	57 100			

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February, 1904	46 300	2 670 000
March, 1904	73 300	4 510 000
February, 1907	45 700	2 540 000
March, 1907	55 700	3 420 000
March 18th–21st, 1907	152 000	1 200 000

* Maximum stage, 29.4 ft.; discharge, 204 000 cu. ft. per sec. at 2 P. M.

† February 16th, the stage was 28.00 ft.; maximum stage, 31.0 ft.; discharge, 224 000 cu. ft. per sec. in the evening. On the 15th the stage was 17.4 ft. and on the 17th, 15.2 ft.

The mean rate of flow at this station, for the 24 hours when it was greatest, was 20.7 cu. ft. per sec. per sq. mile, and the mean for the 4 days, March 18th-21st, was 16.3 cu. ft. per sec. per sq. mile.

Attention is directed to the fact, shown in Table 7, that, although the flow for 4 days of the 1907 flood was greater than for any 4 consecutive days of 1904, the total flow for March, 1907, is only 76% of that for March, 1904.

By comparing the discharges and drainage areas above the gauging stations on the Pit, McCloud, and Sacramento Rivers, it is seen that, although the drainage area above the stations on the Pit and McCloud Rivers is 38% of that above the station on the Sacramento, the combined flow at these two stations is only 21% of that of the Sacramento during February, and 23% during March. This condition is due largely to the slower melting of snow during these months in the higher parts of the basin.

Feather River.—The Feather, the largest tributary of the Sacramento, derives its water from melting snow in the high Sierras, the highest point in its basin being more than 10 000 ft. above sea level. The main river is formed by the union of three streams, the North, Middle, and South Forks, above Oroville. Its principal tributaries are the Yuba and Bear Rivers, which enter it below Oroville.

TABLE 8.—FLOW OF FEATHER RIVER, AT OROVILLE.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.	Date, 1904.	Discharge, in cubic feet per second.
March 16th	19.7	64 000	March 17th	52 000
" 17th	22.2	79 200	" 18th	95 000
" 18th	26.75	107 900	" 19th	88 000
" 19th	30.2*†	129 600	" 20th	82 000
" 20th	23.1*	84 900	" 21st	59 550
" 21st	20.2*	66 740	" 22d	43 350
" 22d	17.3*	49 300	" 23d	35 950
" 23d	16.5*	44 750	" 24th	29 900
" 24th	14.3*	32 650	" 25th	25 100
" 25th	13.5*	28 500	" 26th	23 250
" 26th	12.9*	25 550		

Period.	Mean daily discharge, in cubic feet per second	Total run-off, in acre feet.
February, 1904	27 800	1 600 000
March, 1904	39 500	2 430 000
February, 1907	21 500	1 190 000
March, 1907	36 000	2 210 000
March 18th-21st, 1907	97 300	770 000

* Estimated. † Maximum, about 1 A. M., 185 000 cu. ft. per sec.

The gauging station is on the main stream, in the foot-hills at Oroville. The drainage area above it is 3 640 sq. miles. Table 8 contains data on the flood flow at this station during the floods of 1907 and 1904.

The mean rate of flow for the 24 hours when it was greatest was 35.6 cu. ft. per sec. per sq. mile, and the mean for the 4 days, March 18th-21st, was 26.7 cu. ft. per sec. per sq. mile.

It is seen that, although the maximum daily discharge in 1904 is only 73% of that in 1907, the total flow for February and March is greater in 1904 than in 1907 by 29% and 10%, respectively.

The other four largest floods in the stream on record, or even recalled by the oldest inhabitants living along it, occurred in 1849, 1853, 1861, and 1881. In none of these floods, however, was the water as high at Oroville as in March, 1907. This may have been due in part or entirely to the filling of the river channel at and below Oroville with mining débris. About one-half of the Town of Oroville was flooded for 3 days. The water was about 3 ft. deep on the floor of the Union Hotel. The highway bridge and the Northern Electric Railway bridge in Oroville were swept away, and also other bridges along this stream.

The great range of river stage and its rapid fluctuations are shown by Table 9, the gauge record at Big Bend, 15 miles above Oroville.

TABLE 9.—GAUGE RECORD ON FEATHER RIVER, AT BIG BEND.*

Day.	Hour.	Gauge height.
March 14th	7 A. M.	6.9
" 15th	8 "	6.6
" 16th	8 "	6.6
" 17th	9 "	10.0
" 17th	1 P. M.	13.0
" 17th	6 "	20.0
" 18th	3 A. M.	25.0
" 18th	10 "	31.0
" 18th	4 P. M.	32.5
" 19th	1 A. M.	36.0
" 19th	10 "	34.5
" 20th	10 "	28.0
" 21st	10 "	22.0
" 22d	5 P. M.	18.0
" 25th	10 A. M.	10.0

NOTE: Low-water reading, 2 ft.

* Data furnished by Great Western Power Company, through Mr. L. J. Bevan.

Indian Creek.—Indian Creek is a tributary of the North Fork of the Feather River, and its water-shed is at a high altitude. The gaug-

ing station is about 1½ miles below the Town of Crescent Mills. The drainage area above this station is 740 sq. miles, the larger part of it being at an elevation of more than 5 000 ft.

TABLE No. 10.—FLOW OF INDIAN CREEK, NEAR CRESCENT MILLS.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	4.4	905
" 17th	7.1	2 570
" 18th	17.0	9 500
" 19th	19.7*	11 500
" 20th	17.9	10 100
" 21st	14.7	7 890
" 22d	10.95	5 265
" 23d	9.0	3 900
" 24th	7.8	3 060
" 25th	7.7	2 990
" 26th	7.5	2 850

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	2 210	123 000
March	2 940	181 000
March 18th-21st	9 750	77 400

* Maximum, 20.2 ft.

The mean rate of flow for the 24 hours when it was greatest was 15.5 cu. ft. per sec. per sq. mile.

It is seen that the maximum run-off per square mile during this flood is less than one-half of that from the water-shed of the Feather River above Oroville, due to the slower melting of the snow at high altitudes.

Yuba River.—The Yuba is the largest tributary of the Feather River, entering it at Marysville, 30 miles above the junction of the Feather and Sacramento Rivers and 26 miles below Oroville. The entire area drained by it is about 1 330 sq. miles, of which 1 220 sq. miles are above the gauging station near Smartsville. The basin is comparatively long and narrow, the highest point having an elevation of 9 000 ft., which is not as great as that of the Feather River, the highest point of which is more than 10 000 ft. A large part of the basin is more than 5 000 ft. above sea level. Table 11 contains data on the flow of this stream at Smartsville during the floods of 1907 and 1904.

TABLE 11.—FLOW OF YUBA RIVER, AT SMARTSVILLE.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.	Date, 1904.	Discharge, in cubic feet per second.
March 16th.........	14.3	6 600	February 16th.....	58 000
" 17th.........	24.0	56 000	" 17th.....	41 000
" 18th.........	27.9	85 000	" 18th.....	17 880
" 19th.........	29.2*	100 000	" 19th.....	12 340
" 20th.........	24.0	60 000	" 20th.....	9 350
" 21st.........	18.5	27 000	" 21st.....	9 350
" 22d.........	15.9	14 000	" 22d.....	59 800
" 23d.........	16.4	16 500	" 23d.....	27 660
" 24th.........	15.0	11 000	" 24th.....	59 800
" 25th.........	14.5	9 900	" 25th.....	24 080
" 26th.........	14.1	8 900		

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February, 1904.................	14 900	858 000
March, 1904.....................	15 400	947 000
February, 1907.............	14 100	783 000
March, 1907.....................	17 300	1 060 000
March 18th-21st, 1907..........	68 000	537 000

* Maximum stage, 29.5 ft. about 2 P. M.

The mean rate of flow for the 24 hours when it was greatest was 82.0 cu. ft. per sec. per sq. mile. The maximum daily discharge of this stream was 67% greater in 1907 than in 1904. The total discharge for March, 1907, is larger than for March, 1904, but the total for February and March combined is about the same for the two years. The effect of rapid melting of snow in the middle altitudes is clearly shown here by the large run-off per square mile.

Bear River.—The Bear is the most southern tributary of the Feather River, entering it about 12 miles above the mouth. It drains an area of about 290 sq. miles, of which 263 sq. miles are above the gauging station at Van Trent, 8 miles above Wheatland. Its headwaters do not reach back to the crest of the Sierras, and, as much of its drainage basin is deforested, it is more torrential than the main stream. The greatest altitude in the basin is about 5 500 ft. Table 12 contains data on its flow during the 1907 flood.

The mean rate of flow for March 19th is 106.5 cu. ft. per sec. per sq. mile, and the mean for March 17th-20th is 75.3 cu. ft. per sec. per sq. mile.

It will be noticed that the run-off per square mile was 106.5 cu. ft. per sec. on March 19th, and 102.7 cu. ft. per sec. on February 2d.

TABLE 12.—FLOW OF BEAR RIVER, AT VAN TRENT.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	4.9	900
" 17th	13.95	18 200
" 18th	12.75	15 500
" 19th	17.8	28 000
" 20th	13.6	17 400
" 21st	9.6	8 400
" 22d	8.8	6 600
" 23d	13.2	16 500
" 24th	9.7	8 600
" 25th	9.4	8 000
" 26th	8.1	5 000

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	3 460	192 000
March	5 570	342 000
March 17th-20th	19 800	157 000

NOTE: On February 2d the discharge was 27 000 cu. ft. per sec.

American River.—The American River drains an area of about 2 000 sq. miles, directly south of Bear River Basin and north of Cosumnes River Basin. It has three main forks, two of which head at an elevation of about 9 000 ft. above sea level, while the South Fork reaches back to an elevation of more than 9 600 ft. The gauging station is at Fair Oaks, and the drainage area above it is 1 910 sq. miles, a large part of which has an altitude of more than 5 000 ft. Table 13 contains data on the flow of this stream during the flood of 1907.

TABLE 13.—FLOW OF AMERICAN RIVER, AT FAIR OAKS.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.1	6 800
" 17th	13.40	33 000
" 18th	20.60	63 200
" 19th	27.6*†	93 000
" 20th	23.9*	77 000
" 21st	21.0*	65 000
" 22d	18.4*	54 000
" 23d	13.5	33 400
" 24th	13.25	32 300
" 25th	12.30	28 400
" 26th	11.50	25 000

Period.	Mean daily discharge, in cubic feet per second	Total run-off, in acre-feet.
February	14 200	789 000
March	23 200	1 430 000
March 18th-21st	74 600	594 000

* Bridges and gauges washed away. Gauge height estimated.
† Maximum stage, 30.2 ft., about 5 A. M.

The greatest mean daily discharge at this station was 48.7 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 39.1 cu. ft. per sec. per sq. mile.

Attention is called to the fact that the run-off per square mile for the American River is about 50% greater than that of the Feather River, although the percentage of each basin with an altitude exceeding 5 000 ft. is about the same.

Stony, Cache, and Puta Creeks.—These three streams are the largest tributaries of the Sacramento River from the west. They drain the eastern slope of the Coast Range, and are torrential. Stony Creek is the only one of the three that flows directly into the Sacramento River; the other two empty into Yolo Basin. The gauging station on Stony Creek is in the foot-hills, near Fruto, and the drainage area above it comprises 601 sq. miles. The station on Cache Creek is near Yolo, and the area above it is 1 230 sq. miles. Puta Creek station is at Winters, and the area above it is 805 sq. miles. Table 14 contains data on the flow of these streams during the flood of 1907.

TABLE 14.—FLOW OF STONY, CACHE, AND PUTA CREEKS.

Date, 1907.	Stony Creek.		Cache Creek.		Puta Creek.	
	Gauge.	Discharge.	Gauge.	Discharge.	Gauge.	Discharge.
March 17th	9.45	6 100	6.80	2 950	15.30	8 800
" 18th	14.25	25 000	19.45	13 500	21.60	19 800
" 19th	13.15	20 000	25.90†	19 000	23.65	24 700
" 20th	11.8	13 450	18.20	12 500	16.15	10 000
" 21st	9.8	6 810	12.65	7 820	12.35	5 460
" 22d	7.75	3 350	12.00	7 300	11.90	5 000
" 23d	11.55	12 300	20.85	14 800	26.60*	31 500
" 24th	8.7	4 760	19.30	13 400	15.60	9 200
" 25th	8.15	3 910	16.15	10 800	14.75	8 100
" 26th	7.75	3 350	12.55	7 750	11.40	4 500

Period.	Mean daily discharge, in cubic feet per second.			Total run-off, in acre-feet.		
	Stony Cr.	Cache Cr.	Puta Cr.	Stony Cr.	Cache Cr.	Puta Cr.
February	3 330	2 320	1 740	185 000	129 000	96 600
March	4 450	5 310	5 030	273 000	326 000	309 000
March 18th-21st	16 300	13 200	15 000	129 000	105 000	119 000

* Maximum, 28.15 ft., about noon. † Maximum, about 26.4 ft., during night.

The greatest daily rate of flow per square mile was, for Stony Creek, 41.6 cu. ft. per sec.; for Cache Creek, 15.5 cu. ft. per sec.; for Puta Creek, 39.1 cu. ft. per sec. The small run-off from Cache Basin is attributed to the storage and regulation effects of Clear Lake.

San Joaquin River.—Prior to the flood of March, 1907, no gauging station had been established on the San Joaquin River, because of inability to find a satisfactory section. In the fall of 1907, however, a station was established in the foot-hills near Pollasky, about 20 miles northeast of Fresno, where fair conditions obtain. The drainage area above this station is 1 640 sq. miles. In making an estimate of the run-off for March 18th-21st, a rate of 10 cu. ft. per sec. per sq. mile has been used. This rate is based on the rates in the basins to the north and south, and is believed to be quite conservative.

Mokelumne River.—Mokelumne River has a very narrow and very long drainage basin which extends eastward to the summit of the Sierras. From the low foot-hills to the junction of the three branches, about 30 miles above, its basin is a broad canyon with a minimum width of 1.1 miles. A large percentage of the upper basin ranges from 7 000 to 9 000 ft. in elevation, and several peaks are more than 10 000 ft. high.

TABLE 15.—FLOW OF MOKELUMNE RIVER, NEAR CLEMENTS.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	7.15	2 000
" 17th	13.60	8 600
" 18th	17.00	12 200
" 19th	21.00	17 000
" 20th	17.9	13 000
" 21st	15.9	11 200
" 22d	13.0	8 000
" 23d	13.3	8 300
" 24th	13.0	8 000
" 25th	11.6	6 500
" 26th	11.2	6 400

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	2 920	162 000
March	5 320	327 000
March 18th-21st	13 350	106 000

This river drains an area of about 660 sq. miles between the American and Stanislaus Rivers, and empties into the San Joaquin

through Georgiana Slough at the west end of Bouldin Island. It heads in the Sierras at an elevation of about 10 000 ft. The area above the gauging station, near Clements, is 642 sq. miles, fully two-thirds of which exceeds an altitude of 5 000 ft. Its principal tributary is Cosumnes River, which enters from the north, about 8 miles from Walnut Grove, and about 25 miles below the gauging station. Table 15 shows the flow of this stream during the flood of 1907.

The greatest run-off at this station in 24 hours during this flood was 26.5 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 20.8 cu. ft. per sec. per sq. mile.

Attention is called to the fact that the maximum daily discharge was not much larger than the mean for 4 days, showing quite clearly the effect of the configuration of the water-shed. The small run-off per square mile resulted from the fact that such a large percentage of the area is at a high altitude.

Cosumnes River.—Cosumnes River, the chief tributary of the Mokelumne, rises at an elevation of about 7 700 ft. above sea level, and drains an area of about 580 sq. miles. The discharge at Michigan Bar, 30 miles above the mouth, where the drainage area is 524 sq. miles, for the period, March 18th-22d, is given in Table 16.

TABLE 16.—FLOW OF COSUMNES RIVER, AT MICHIGAN BAR.

Date, 1907.	Discharge, in cubic feet per second.
March 18th	7 600
" 19th*	32 600
" 20th	9 300
" 21st	3 900
" 22d	3 300
March 18th-21st	Mean......13 350

* Maximum at 6 A. M.

The greatest run-off in 24 hours during this flood was 62.2 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 25.5 cu. ft. per sec. per sq. mile. The discharge for this stream has been computed from data on the cross-section and slopes of the bed and flood plane obtained after the flood. A regular gauging station has been established at Michigan Bar since the March flood.

Calaveras River.—The Calaveras is a comparatively small stream, draining an area of 490 sq. miles on the western slope of the Sierras

between Mokelumne and Stanislaus Rivers. It empties into the San Joaquin River about 5 miles northwest of Stockton. Its headwaters have an elevation of 6 000 ft., which is greater than those of Bear River, but only a very small portion of the basin exceeds 4 000 ft. The data in Table 17 were obtained during the 1907 flood, near Jenny Lind, above which point the area of the drainage basin is 395 sq. miles.

TABLE 17.—FLOW OF CALAVERAS RIVER, NEAR JENNY LIND.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 18th	5.4	3 800
" 19th	11.4*	26 100
" 20th	5.4	3 800
" 21st	5.0	3 300
" 22d	5.0	3 300
March 18th-21st		Mean....9 250

* Maximum stage at 8 A. M.

The maximum run-off in 24 hours was 66.2 cu. ft. per sec. per sq. mile, and the mean for the 4-day period was 23.4 cu. ft. per sec. per sq. mile.

As in the case of the Cosumnes, the discharge of this stream has been computed from meter measurements after the flood, and from slope and cross-section data. A gauging station has been established at Jenny Lind since the March flood. The small maximum daily run-off of the Mokelumne, as compared with that of the Cosumnes and Calaveras, is quite fully accounted for by the difference in the topography of the basins.

Stanislaus River.—Stanislaus River, which drains the area between Mokelumne and Tuolumne Rivers, empties into the San Joaquin River about 11 miles south of Lathrop. Its headwaters have an elevation of from 10 000 to 12 000 ft. The area drained includes about 1 050 sq. miles, of which 935 sq. miles are above the gauging station at Knights Ferry. About 500 sq. miles have an altitude of more than 5 000 ft. Table 18 contains data on the flow at this station during the flood of 1907.

The greatest run-off in 24 hours at this station during the 1907 flood was 58.1 cu. ft. per sec. per sq. mile, and in 1904 it was 32.3 cu. ft. per sec. per sq. mile. The greatest 4-day mean in 1907 was 34.5

cu. ft. per sec. per sq. mile. It is seen that the maximum run-off at this station in 1904 was only 57% of that in 1907.

TABLE 18.—FLOW OF STANISLAUS RIVER, AT KNIGHTS FERRY.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	8.85	2 730
" 17th	14.50	15 580
" 18th	17.35	24 100
" 19th	25.30	54 300
" 20th	19.10	31 400
" 21st	15.60	19 200
" 22d	14.55	15 740
" 23d	14.15	14 470
" 24th	13.80	13 430
" 25th	14.20	14 630
" 26th	12.75	10 420

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	3 440	191 000
March	9 880	608 000
March 18th-21st	32 250	256 000

TABLE 19.—FLOW OF TUOLUMNE RIVER, AT LAGRANGE.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	6.55	3 420
" 17th	11.20	20 200
" 18th	13.50	33 400
" 19th	15.75	51 800
" 20th	13.00	30 500
" 21st	11.50	21 500
" 22d	10.50	16 700
" 23d	9.80	13 500
" 24th	10.65	17 000
" 25th	10.65	17 000
" 26th	9.30	11 500

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	3 910	217 000
March	11 100	682 000
March 18th-21st	34 300	271 000

Tuolumne River.—Tuolumne River, which drains an area immediately south of the Stanislaus River, heads in the high peaks of the Sierras above Yosemite National Park, at an elevation of about 13 000 ft., and empties into the San Joaquin River about 10 miles west of

Modesto. The area above the gauging station at LaGrange is 1 500 sq. miles. Table 19 contains data on the flow at this station during the 1907 flood.

The greatest daily rate of run-off at this station during this flood was 34.5 cu. ft. per sec. per sq. mile, and the greatest 4-day mean rate was 22.9 cu. ft. per sec. per sq. mile.

Merced River.—Merced River drains the area between Tuolumne River and the Upper San Joaquin, and empties into the latter about 26 miles northwest of Merced. It heads at the summit of Mt. Lyell, at an elevation of 13 090 ft., and drains the southern and western slopes of this mountain, while the Tuolumne drains the northern slope. In this basin is the famous Yosemite Valley, with its great waterfalls and barren domes. The gauging station on this stream is at Merced Falls, above which the drainage area is 1 090 sq. miles. Table 20 contains data on the flow of this stream at the station during the flood of 1907.

TABLE 20.—FLOW OF MERCED RIVER, AT MERCED FALLS.

Date, 1907.	Gauge height.	Discharge, in cubic feet per second.
March 16th	10.85	2 200
" 17th	15.2	14 400
" 18th	14.8	13 000
" 19th	18.0	23 000
" 20th	16.05	17 400
" 21st	14.8	13 000
" 22d	13.95	10 200
" 23d	13.55	8 800
" 24th	16.55	19 200
" 25th	15.60	15 800
" 26th	13.65	9 200

Period.	Mean daily discharge, in cubic feet per second.	Total run-off, in acre-feet.
February	1 920	107 000
March	7 170	441 000
March 18th-21st	16 600	132 000

The greatest daily rate of flow during this flood was 21.1 cu. ft. per sec. per sq. mile, and the greatest 4-day mean was 15.7 cu. ft. per sec. per sq. mile. The small run-off per square mile arises from the fact that much of the basin has a high altitude, and that the precipitation was not as heavy as in the basins to the north.

Flow through Sacramento and San Joaquin Valleys.

The rate of inflow into the Sacramento and San Joaquin Valleys from the metered mountain and foot-hill areas during this flood can be seen from the preceding pages. For ready reference, however, these rates of inflow at gauging stations for the 4-day period, March 18th-21st, are given in Table 21.

TABLE 21.—Run-Off from Sacramento and San Joaquin Basins, in Cubic Feet per Second, for March 18th–21st, 1907.

Stream.	Place.	Drainage, in square miles	Date, March, 1907.				Mean for March 18th-21st.
			18th.	19th.	20th.	21st.	
Sacramento	Red Bluff	9 300	118 000	164 000	192 000	132 000	151 500
Stony	Fruto	601	25 000	20 000	13 450	6 800	16 310
Feather	Oroville	3 640	107 900	129 600	84 900	66 740	97 290
Yuba	Smartsville	1 220	85 000	100 000	60 000	27 000	68 000
Bear	Van Trent	263	15 500	28 000	17 400	8 400	17 300
American	Fair Oaks	1 910	63 200	93 000	77 000	65 000	74 600
Cache	Yolo	1 230	13 500	19 000	12 500	7 820	13 200
Puta	Winters	805	19 800	24 700	10 000	5 460	15 000
Unmetered mountain and foot-hills.		3 907	....	....	....	....	76 000*
Sacramento Valley		4 250	....	....	....	....	25 500‡
Total, Sacramento Basin		27 126	....	....	....	....	554 700
Cosumnes	Michigan Bar	524	7 600	32 600	9 300	3 900	13 350
Mokelumne	Clements	642	12 200	17 000	13 000	11 200	13 350
Calaveras	Jenny Lind	395	3 800	26 100	3 800	3 300	9 250
Stanislaus	Knights Ferry	935	24 100	54 300	31 400	19 200	32 250
Tuolumne	LaGrange	1 500	33 400	51 800	30 500	21 500	34 300
Merced	Merced Falls	1 090	13 000	23 000	17 400	13 000	16 600
San Joaquin	Pollasky	1 640	....	....	....	....	16 400‖
Unmetered mountain and foot-hills.		5 656	....	....	....	....	67 900†
San Joaquin Valley		5 890	....	....	....	....	23 560§
Total, San Joaquin Basin		18 272	....	....	....	....	226 960

* Run-off per square mile assumed as 50% of precipitation for period, March 17th-20th, or 20 cu. ft. per sec.

† Run-off per square mile assumed as 50% of precipitation for period, March 17th-20th, or 12 cu. ft. per sec.

‡ Run-off per square mile assumed as 40% of rainfall for period, March 17th-20th, or 6 cu. ft. per sec.

§ Run-off per square mile assumed as 40% of rainfall for period, March 17th-20th, or 4 cu. ft. per sec.

‖ Run-off per square mile assumed as 10 cu. ft. per sec.

From Table 21 it is seen that the mean rate of run-off from the metered area of the Sacramento Basin (83% of all mountains and foot-hills) for the 4-day period, March 18th-21st, was about 453 000 cu. ft. per sec. The estimated run-off for this period was 76 000 cu. ft. per sec. from the unmetered mountains and foot-hills, and 25 500

cu. ft. per sec. from Sacramento Valley, making a mean rate of run-off from the Sacramento Basin of about 555 000 cu. ft. per sec. for 4 consecutive days.

It is not possible to trace the movement of this water through the valley, on account of overflow into flood basins and breaks in the levee system. The levees failed at many places on both sides of the Sacramento River, and also on some of its tributaries, and it is impossible to compute the flow through any of these breaks. Such an estimate, if correctly made, would have practically no value, as it would give little idea of the distribution of flow through the valley during any other flood when failure of levees occurred at other places.

While an estimate of the volume passing specified places in the valley at a given time cannot be made, the points where large volumes left the channel and returned to it again or crossed it can be indicated, as well as the time of failure of important levees. On the evening of March 20th the water was overtopping the levees for almost the entire distance between Princeton and Jacinto, and also above and below Colusa. On March 21st, eleven breaks in the levees occurred between Colusa and Grimes, and during that night several breaks occurred in the levees on the east side of Sacramento River between Clarksburg and Courtland, allowing water from the Sacramento to pass into Mokelumne River and thence into the San Joaquin. On March 22d several other breaks occurred in Colusa County, and also in the Island District, where large areas of reclaimed land were submerged. On March 23d the levees of Ryer, Tyler, Brannan, Andrus, and Bouldin Islands and the Lisbon District failed, flooding 65 000 acres of land. Besides the failures already mentioned, there were numerous others of more or less seriousness in different places in the Sacramento Valley.

At Knights Landing, on March 21st, the Sacramento was 1 ft. higher than recorded at any previous time. Below this point, a large part of the water from the Feather River was flowing across the Sacramento Channel into Yolo Basin. Through the Kripp crevasse of February 8th, opposite the City of Sacramento, a large part of the waters of the Sacramento and American Rivers also passed into Yolo Basin, and the water level of this basin was several feet higher than ever known before. On February 24th the Sacramento at Rio Vista reached its greatest height during the flood, being 3 ft. higher than

indicated by previous records. The failure of the levees of Brannan, Twitchell, and Andrus Islands, near the mouth of Cache Slough, permitted a part of the water of Yolo Basin to flow across the Sacramento Channel into the San Joaquin River, submerging large areas in the San Joaquin Delta. In all, it is estimated that about 300 000 acres of reclaimed land were submerged during this flood. Below the City of Sacramento, the only reclaimed districts having levees that withstood the high waters are: Reclamation District No. 744; Merritt Island, Grand Island, and Randall Island Reclamation Districts; Geo. W. Locke, private reclamation; Reclamation District No. 545; Sutter and Sherman Islands; and the northern portion of Union Island.

Referring again to Table 21, it is seen that, in all the streams of the San Joaquin Basin, the greatest rate of flow occurred on March 19th. On this date the mean rate of run-off from the metered area (41% of all mountains and foot-hills) was about 205 000 cu. ft. per sec. The rate from the unmetered area must have been at least 84 000 cu. ft. per sec. from mountains and foot-hills and 24 000 cu. ft. per sec. from the valley, making a maximum run-off of about 313 000 cu. ft. per sec. from the San Joaquin Basin. The mean rate for 4 days, March 18th-21st, was about 227 000 cu. ft. per sec. It is impossible to indicate the volume of flow at different points in this valley owing to the failure of levees on both the San Joaquin and Sacramento Rivers, and the passage of a large volume from the latter into the former, producing back-water and retardation of flow.

It is also seen from Table 21 that the mean flow from the mountains and foot-hills of the Sacramento and San Joaquin Basins combined, for the 4 days, March 18th-21st, was about 732 000 cu. ft. per sec. It is seen, too, that the mean rate of discharge into Suisun Bay for these 4 days, if storage in the valleys had not been permitted, would have been about 782 000 cu. ft. per sec., a volume for these 4 days of 6 200 000 acre-ft., or 9 690 mile-ft., enough to cover both basins to a depth of 2.56 in., if spread over them evenly.

Table 22 shows the run-off, expressed as depth, in inches, over the drainage basin, together with the precipitation for the March flood. Of course, there is the very regrettable condition of too few and poorly placed precipitation stations, but it is believed that the records here given are quite representative for the different basins. This table

gives some idea of the effects of altitude and of melting snow in the various drainage areas.

TABLE 22.—RUN-OFF, AS DEPTH, IN INCHES.

Stream.	Place of gauging.	Drainage area above station.	Altitude of source, in feet.	Run-off per Square Mile, March, 1907.				
				Maximum, in cubic feet per second.	Mean for March 18th-21st.			
					Cubic feet per second.	Depth, in inches.	Precipitation, in inches.	Percentage of precipitation.
Pit	Bieber	2 950	9 900	9.3	8.3	1.23	....	....
McCloud	Gregory	608	14 400	50.0	35.5	5.28	....	....
Sacramento	Red Bluff	9 300	14 400	20.7	16.3	2.43	6.56	37
Feather	Oroville	3 640	10 000	35.6	26.7	3.97	10.40	29
Indian Cr.	Crescent Mills	740	7 000	15.5	13.2	1.96	10.00	20
Yuba	Smartsville	1 220	9 000	82.0	55.7	8.29	10.33	80
Bear	Van Trent	263	5 500	106.5	75.3	11.30	8.13	139
American	Fair Oaks	1 910	9 600	48.7	39.1	5.81	8.63	67
Stony Cr.	Fruto	601	....	41.6	27.1	4.03	5.27	76
Cache Cr.	Yolo	1 230	....	15.5	9.3	1.38	5.00	28
Puta Cr.	Winters	805	....	39.1	18.6	2.77	5.10	54
Cosumnes	Michigan Bar	524	7 700	62.2	25.5	3.80	7.50	51
Mokelumne	Clements	642	10 000	26.5	20.8	3.08	6.42	48
Calaveras	Jenny Lind	395	6 000	66.2	23.4	3.48	5.06	69
Stanislaus	Knights Ferry	935	11 500	58.1	34.5	5.14	6.26	82
Tuolumne	LaGrange	1 500	13 000	34.5	22.9	3.40	6.65	51
Merced	Merced Falls	1 090	13 000	21.1	15.7	2.34	5.92	40
San Joaquin	Pollasky	1 640	13 000	(Est.)	10.0	1.49	5.00	30

RATE OF FLOW IN SACRAMENTO VALLEY.

It will be instructive to compute the probable rate of flow of the Sacramento River during this flood at the four places where it receives large volumes of water from tributaries, namely, just below the mouths of Stony Creek, Feather and American Rivers and Cache Slough, taking into account the time required for the water to pass from the gauging stations to the Sacramento and the time to pass between the above-mentioned places. No great degree of refinement will be attempted, as the data will not warrant it.

As a flood wave travels down a channel there is a gradual diminution of its height, due to the filling of the channel and the flattening of the wave. Such diminution would have been small for this flood, and is neglected in the computations, for the following reasons:

(1).—The flood wave was a long one, the water at some of the stations continuing to rise for 4 days;

(2).—The streams had reached a comparatively high stage on March 17th, and consequently their channels were from more than half to two-thirds full at the date when the computations begin;

(3).—The rates of flow computed at gauging stations are 24-hour means, not maxima for a few hours.

It can be shown that the speed of a flood wave, M, in a stream channel, is given by the equation, $\frac{d\ Q}{d\ h} = M\ W$, in which $d\ Q$ is the increment of discharge corresponding to the increment of stage, $d\ h$, and W is the channel width. The value of M has been computed at each gauging station for intervals of 1 ft. in gauge height during the flood stages, and a mean value obtained for the distance, in hours, from the gauging station to places along the Sacramento River. These results are given in Table 23:

TABLE 23.—DATA ON RATE OF PROGRESS OF FLOOD WAVE, IN STREAMS, IF WATER WERE CONFINED IN CHANNELS.

Place to place.	Distance, in miles.	Rate of travel, in miles per hour.	Time of travel, in hours.
Gauging Station, Sacramento River to mouth of Stony Creek	40	9	5
Gauging Station, Stony Creek to mouth of Stony Creek	35	6	6
Gauging Station, Feather River to mouth of Feather River	60	7	9
Gauging Station, Yuba River to mouth of Feather River	50	8	6
Gauging Station, Bear River to mouth of Feather River	15	5	3
Mouth of Stony Creek to mouth of Feather River	100	7	14
Gauging Station, American River to mouth of American River	15	5	3
Mouth of Feather River to mouth of American River	20	7	3
Mouth of American River to mouth of Cache Slough	46	7	7
Gauging Station, Cache Creek to mouth of Cache Slough	45	4	11
Gauging Station, Puta Creek to mouth of Cache Slough	45	4	11

NOTE: The rate of travel for flood waves, as given above, is the mean of the computed rates on each of the days, March 17th-21st, reduced, in most instances, by a considerable percentage.

A study of the daily rate of discharge of the streams in the Sacramento Basin, for March 18th-21st, Table 21, shows that the discharge at places along the Sacramento River was undoubtedly at a maximum when the crest of the wave from the Feather River reached them. This wave crested at Oroville about 1 A. M., March 19th. As Oroville is about 9 hours above the mouth of Feather River, the crest would reach the Sacramento River at about 10 A. M., March 19th, with a discharge of about 258 000 cu. ft. per sec., including the Yuba and

Bear Rivers. This amount, combined with the flow in the Sacramento at that time, would give the maximum discharge just below the mouth of the Feather River. The flow in the Sacramento at this time, however, was the flow at the gauging station above, about 19 hours before, combined with the flow at the gauging station on Stony Creek, about 20 hours before, or the flow of the two at, say, 2 P. M., March 18th. This flow was 143 000 cu. ft. per sec., which, added to the 258 000 cu. ft. per sec. from the Feather River, would give a discharge of 401 000 cu. ft. per sec. in the Sacramento. This volume would reach the mouth of American River 3 hours later, and be augmented by 93 000 cu. ft. per sec. passing the gauging station 3 hours before, so that the maximum discharge in the Sacramento below the mouth of the American River would be about 494 000 cu. ft. per sec., and would occur at about 1 P. M., March 19th. This volume would reach the mouth of Cache Slough at about 8 P. M., March 19th, to be increased by the flow of the Cache and Puta Creeks at the gauging stations 11 hours before, which amounted to about 44 000 cu. ft. per sec. Below the mouth of Cache Slough, therefore, the discharge would have been about 538 000 cu. ft. per sec. It is to be noted that the maximum flow in the Sacramento below the mouth of Stony Creek was about 205 000 cu. ft. per sec., and did not occur until some time on March 20th.

The figures just given do not include the unmetered flow of 76 000 cu. ft. per sec. from the mountains and hills below the metered basins, nor the 25 500 cu. ft. per sec. from the valley. It is evident that, unless stored in the flood basins, it must have appeared in the Sacramento below Cache Slough. It is impossible to compute the increase in discharge at the different places on the Sacramento River due to these two rates of inflow, because it is not known at what points all these waters were delivered; but it is quite clear that there must have been a very decided increase above the mouth of Stony Creek from each side of the river. On the east side there are 1 600 sq. miles of mountains and foot-hills lying between the Feather and Upper Sacramento Basins, which are drained by numerous creeks, the most important of which are Mill and Deer Creeks, the headwaters of which come from Lassen Peak, more than 10 000 ft. in altitude. Several of the stations reporting the greatest precipitation in March, 1907, are in this area or very near it. Taking into consideration its position between two

basins in which the rate of run-off is known, together with its heavy precipitation and generally lower altitude, it is believed that the mean rate of run-off may be safely placed at 25 cu. ft. per sec. per sq. mile for the period, March 18th-21st. This means 40 000 cu. ft. per sec. from this side. On the west, above the Stony Creek Basin, are 1 080 sq. miles of mountains and foot-hills, for which it is safe to put the run-off at 15 cu. ft. per sec. per sq. mile, or a mean of 16 000 cu. ft. per sec. for March 18th-21st. This would mean an increase in the discharge below Stony Creek of about 56 000 cu. ft. per sec.

A considerable area of mountains and foot-hills between the Feather and Bear Basins must have contributed a large volume to the Sacramento through the Feather River, so that, all told, the maximum discharge below the mouth of the Feather River was probably at least 65 000 cu. ft. per sec. greater than that computed above. As the rates of run-off for the unmetered area of mountains and valley are 4-day means, the maximum discharge below Cache Slough must have been about 640 000 cu. ft. per sec. This maximum, however, is only 15% greater than the 4-day mean flow of 555 000 cu. ft. per sec. for March 18th-21st.

It will be noticed that the maximum discharge just below the mouth of Cache Slough would probably occur at 8 P. M., March 19th, if the water were confined in channels. But the maximum stage at Rio Vista, a few miles below the mouth of this slough, actually occurred at 11 P. M., March 23d. Overflow and storage in the flood basins, therefore, delayed the arrival of the flood crest at Cache Slough about 4 days.

Table 24 is a comparison of maximum rates of flow of the Sacramento River during this flood with those assumed by the 1904 Engineering Commission, provided that the total run-off is confined between the levees and not allowed to collect in the flood basins.

TABLE 24.

Place.	Maximum rate assumed by 1904 Engineering Commission. Cubic feet per second.	Maximum rate computed from March, 1907, flood. Cubic feet per second.
Below mouth, Stony Creek.....	180 000	261 000
" " Feather River..	190 000	466 000
" " American River.	230 000	559 000
" " Cache Slough...	250 000	640 000

These computed rates are from 45 to 156% larger than the assumed rates.

Profile of Flood Wave in Sacramento River.

Fig. 2 is a profile of the flood wave in Sacramento River during March, 1907. This profile merely shows the greatest elevation of the flood plane above mean sea level at different points along the course of the river. In other words, the maximum height attained by the flood at various points is platted with reference to the distance from the mouth of the river and the elevation above mean sea level. An inspection of this profile shows that the mean gradient of the flood plane, in feet per mile, between observed points, decreases quite rapidly from Red Bluff toward the mouth of the river, actually changing sign below Walnut Grove. This gradient varies from — 2.41 between Red Bluff and Munroeville, near the mouth of Stony Creek, to + 0.01 below Walnut Grove. Such a reversal of slope would seem to indicate a constricted condition of the channel near the mouth of the river.

A profile of the flood wave of 1905, made under the direction of the Commissioner of Public Works of California, is also shown on Fig. 2 for the purpose of comparison. This profile may be considered as typical of the usual flood wave in the spring of each year.

Losses Due to the Flood of March, 1907.

The losses resulting from this flood consisted mainly in the destruction of the crops then growing on about 300 000 acres of land completely inundated, together with the damage done to a portion of the prospective yield for the season of 1907. In addition to this, many miles of costly levees had to be rebuilt and many miles more extensively repaired on account of overtopping and wind action. The railroads suffered heavily, in bridges and culverts washed out, in injury to miles of roadbed, and in loss of traffic. The line from Marysville to Knights Landing was closed from March 19th to May 13th. Among the larger bridges swept away or badly damaged were the highway and the Northern Electric Railway bridges across the Feather River at Oroville, the highway bridge across the American River at Fair Oaks, the highway bridge on the Mokelumne River near Clements, and the bridge on the Cosumnes River at Bridge House. Three costly dredges for mining gold-bearing gravel in the Feather River near Oroville were

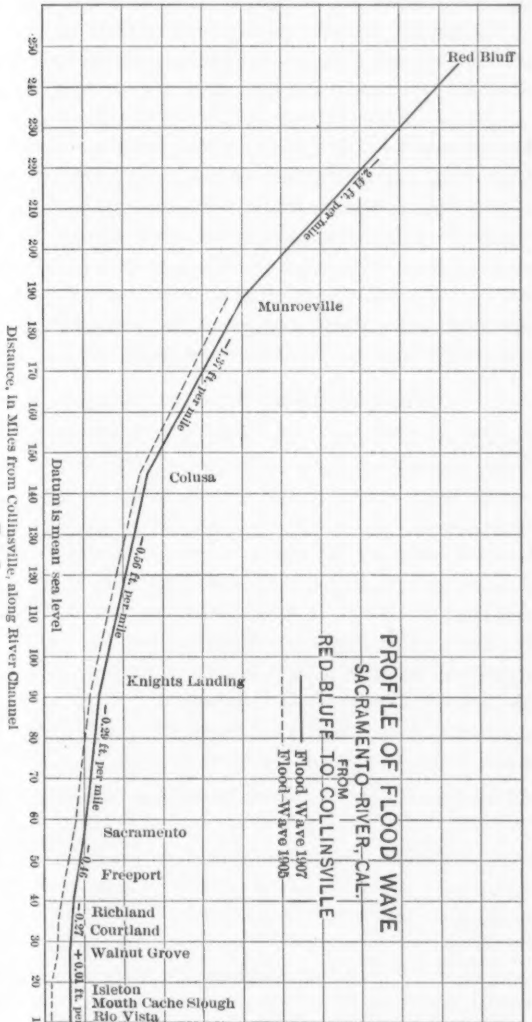

PROFILE OF FLOOD WAVE
SACRAMENTO RIVER, CAL.
FROM
RED BLUFF TO COLLINSVILLE
Flood Wave 1907
Flood Wave 1905
Red Bluff
2.41 ft. per mile
Munroeville
1.37 ft. per mile
Colusa
0.56 ft. per mile
Knights Landing
0.29 ft. per mile
Sacramento
Freeport
0.46
Richland
Courtland
0.27
Walnut Grove
+0.01 ft. per mile
Isleton
Mouth Cache Slough
Rio Vista
Collinsville
Elevations in feet above mean sea level
Datum is mean sea level
Distance, in Miles from Collinsville, along River Channel
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240
250
0
25
50
75
100
125
150
175
200
225
250
275
300

Fig. 2.

destroyed. Many towns and villages were partially inundated, subjecting the inhabitants to serious inconvenience at the time and to heavy expense in repairs later. The greater part of Stockton was flooded for nearly a week, because of the failure of the levees along Mormon Slough and Jackson Creek. About half of Oroville was flooded for three days, and one hundred and twenty-five families were driven from their homes. The restraining dam on the Yuba River, 14 miles above the mouth, known as Barrier No. 1,* was destroyed on the night of March 18th. This dam was built to hold back the mining débris in the channel above. With its destruction, practically all the débris restrained by it (probably amounting to more than 1 000 000 cu. yd.), was transferred to the channel below. It is estimated that the total damage resulting from this flood exceeded $5 000 000.

Effect of Mining Débris on Floods.

From 1849 to 1880 enormous quantities of débris—sand, gravel, and cobbles, the tailings from hydraulic mining—were deposited in the upper course of several of the streams on the eastern slope of the Sacramento Basin. The volume of this débris in the Yuba River alone has been variously estimated at from 71 000 000 to 700 000 000 cu. yd. At the mouth of the river, near Marysville, it has a depth of 7½ ft.; at Dugnens Point, 11 miles above the mouth, it has a depth of 26 ft., and at The Narrows, 18 miles above the mouth, it has a depth of 84 ft. The gradual elevation of the flood plane at Marysville, due to the accumulation of débris in the channel at this place, is shown by the maximum gauge readings at Marysville (Table 26). The zero of the gauge is the elevation of low water in 1872.

TABLE 26.—Maximum Gauge Readings at Marysville.

Date.	Gauge height.†
January 11, 1862	11 ft. 6 in.
March 6, 1869	15 " 11 "
January 19, 1875	15 " 2 "
April 22, 1880	18 " 2 "
February 24, 1881	18 " 2 "
December 23, 1884	17 " 1 "
January 18, 1896	18 " 5 "
March 25, 1899	18 " 5 "
February 21, 1901	19 " 0 "
February 25, 1904	20 " 0 "
January 19, 1906	21 " 8 "
February 2, 1907	21 " 3 "
March 19, 1907	23 " 4 "

* The failure of this structure is described in *Engineering News*, Aug. 8th, 1907.
† Data furnished by W. T. Ellis, Levee Commissioner.

The low-water reading on this gauge, in the summer of 1906, was 9.0 ft. The flood of 1907, however, changed the low-water channel from the right to the left side of the river, so that in August the elevation of the water surface could not be read at all, the débris around the gauge being 3 ft. higher than the water.

The failure of Barrier Dam No. 1, on the Yuba River 14 miles above the mouth, liberated about 1 300 000 cu. yd. of débris which was deposited in the bed of the stream at varying distances below the dam, depending upon the size of the material. The deposition of this enormous volume of material in the stream bed, and the gradual elevation of the flood plane due to it, require frequent raising and widening of the levees along the river. Such a condition is fraught with growing peril to the valley land and to all interests adjoining the river.

Effect of Storage Reservoirs on Floods.

Any rational system of reclamation for the overflow lands in the Sacramento and San Joaquin Valleys must make provision for passing the peak of the floods rapidly to Suisun Bay. The volume of flood water to be passed in Sacramento Valley, as determined by actual gaugings of the flood of March, 1907, largely exceeds all estimates previously used as a basis for the computation of proper channel capacity to carry safely the flood waters of the Sacramento River. Indeed, it may be that the task of rectification and enlargement of channel necessary to pass such floods as that of March, 1907, is so great as to make it economically impossible. In such event, some auxiliary system of flood control would have to be devised. Probably no more effective and easily executed auxiliary system could be found than that of large, regulating storage reservoirs in the mountains. Such reservoirs could be utilized to store water during floods, thereby reducing the peak of the flood in the valley sufficiently to allow the main channel to carry it safely to Suisun Bay.

The United States Reclamation Service has located the principal reservoir sites in the Sacramento Basin, and has made surveys to determine the capacity and probable cost of most of them. Of the reservoirs surveyed to date, four are in Stony Creek Basin, with a total capacity of 124 100 acre-ft.; two are in Cache Creek Basin, with a total capacity of 176 500 acre-ft.; two are in Puta Creek Basin, with a total capacity of 318 000 acre-ft.; seven are in Feather River Basin,

with a total capacity of 775 600 acre-ft.; four are in Pit River Basin, one of which has a capacity of 3 196 000 acre-ft.; and one is on the Upper Sacramento River at Iron Canyon, with a capacity of 226 900 acre-ft. In the San Joaquin Basin no reservoir sites have been located and surveyed yet, although it is probable that the area contains some good ones.

TABLE 27.—RESERVOIR DATA.

Name of Reservoir.	Capacity, in acre-feet.	Drainage, in square miles.	Volume Available for Storage, in Acre-Feet.*				
			Mar. 18th.	Mar. 19th.	Mar. 20th.	Mar. 21st.	Mar. 18th–21st.
Stony Creek Basin.							
East Park	26 000	114	9 390	7 520	5 060	2 560	24 530
Stony Ford	40 000	110	9 060	7 250	4 880	2 470	23 660
Briscoe	14 400	58	4 770	3 820	2 570	1 300	12 460
Mill Site	43 700	323	26 600	21 300	14 300	7 250	69 450
Cache Creek Basin.							
Clear Lake	100 000	486	10 600	14 900	9 790	6 120	41 410
Little Indian	76 500	123	2 680	3 770	2 480	1 550	10 480
Below reservoirs and above gauging station	....	621	13 500	24 600	12 500	7 820	52 820
Puta Creek Basin.							
Guenoc	188 000	148	7 210	8 990	3 460	1 990	21 830
Puta Creek	130 000	603	29 400	36 600	14 800	8 110	88 910
Below reservoirs and above gauging station	....	54	1 980	2 460	998	546	5 984
Feather River Basin.							
Grizzly Valley	61 800	44	2 580	3 100	2 040	1 600	9 320
Mohawk Valley	12 600	682	40 000	48 100	31 700	24 800	144 600
Big Meadow	500 000	506	29 700	35 600	23 500	18 400	107 200
Buck's Valley and Spanish Ranch	46 270	29	1 700	2 040	1 350	1 050	6 140
American Valley	86 100	172	10 100	12 100	8 000	6 260	36 460
Indian Valley	68 800	1 010	59 300	71 200	47 000	36 700	214 200
Below reservoirs and above gauging station	....	1 200	70 400	84 500	55 800	43 600	254 300
Pit River Basin.							
Big Valley	3 196 000	2 950	49 600	54 500	49 600	41 200	194 900
Sacramento River.							
Iron Canyon	226 900	6 350	184 400	270 500	331 400	220 800	1 007 100

* The daily run-off per square mile is assumed to be constant over the basin above the gauging station.

In Table 27 are shown the reservoir sites in the Sacramento Basin which could be used for flood control, together with the drainage area

tributary to each and its capacity in acre-feet. Assuming the run-off per square mile to be constant in any particular basin, the quantity of water available for storage at each reservoir is given for each of the days, March 18th-21st, and also the total for the 4 days. It will be noted that some of these reservoirs would be only partially filled by the flood flow of March 18th-21st, while others would store but a small percentage of the run-off for this period.

A study of Table 27 will show that the four reservoirs in Stony Creek Basin would have stored the run-off from 481 sq. miles, or 80% of the area above the gauging station, and would have reduced the maximum daily flow from 25 000 to 5 000 cu. ft. per sec. The two reservoirs in Cache Creek Basin would have stored the flow from 609 sq. miles, or 50% of the area above the gauging station, and would have reduced the maximum daily flow from 19 000 to 9 500 cu. ft. per sec. The two reservoirs in Puta Creek Basin would have stored the flow from 751 sq. miles, or 93% of the area above the gauging station, and would have reduced the maximum daily flow from 24 700 to 1 700 cu. ft. per sec.

The seven reservoirs in Feather River Basin would have stored the flow from about 1 134 sq. miles, or 31% of the area above the gauging station at Oroville, leaving 2 506 sq. miles uncontrolled. Of this uncontrolled area, 623 sq. miles are above Mohawk Valley Reservoir, 683 sq. miles are above Indian Valley Reservoir, and 1 200 sq. miles are below the reservoirs and above the gauging station. This storage would have reduced the daily flow at Oroville as follows:

From 107 900 to 74 300 cu. ft. per sec. on March 18th; from 129 600 to 89 200 cu. ft. per sec. on March 19th; from 84 900 to 58 500 cu. ft. per sec. on March 20th; and from 66 740 to 45 900 cu. ft. per sec. on March 21st. Big Valley Reservoir, on Pit River, would have stored the entire flow at that place and reduced the daily flow of the Sacramento River at Red Bluff about 25 000 cu. ft. per sec. The storage at Iron Canyon, together with that on Pit River, would have reduced the greatest daily flow of the Sacramento River at Red Bluff from 192 000 to 106 000 cu. ft. per sec.

The combined effect of all these reservoirs in operation at the same time would have been to reduce the maximum flow in the Sacramento River by about 86 000 cu. ft. per sec. above the mouth of Stony Creek, 106 000 cu. ft. per sec. above the mouth of the Feather River, and 179 000 cu. ft. per sec. below the mouth of Cache Slough.

It would seem that the ultimate solution of the flood problem in the lower portions of the Sacramento Valley is closely interwoven with the reclamation of the higher portions by irrigation. Reservoirs which would impound flood waters and reduce the peak of floods, so as to save the lowlands from overflow in the early spring, would serve later as storage reservoirs from which to draw for irrigation purposes. The flood problem in this valley is indeed a very serious one, and merits the most careful and thoughtful consideration.

DISCUSSION.

C. E. Grunsky, M. Am. Soc. C. E. (by letter).—The United States Mr. Grunsky. Geological Survey, through its Water Resources Branch, has again taken up an important subject in a practical way. The engineer who is called upon to improve the navigability of a river, or to improve it as a drainway, or otherwise to rectify it, rarely has opportunity to collect the data relating to stream flow that are necessary for the solution of his problems. Neither time nor opportunity may be at his disposal to observe the river at a critical stage. He must turn to records showing extreme conditions, of which knowledge is an essential requisite, if his conclusions are to be of value. It has been the general experience throughout the United States that the individual States cannot be relied upon to observe continuously rainfall, stream flow, and run-off conditions. This work, therefore, falls naturally to the Federal Government.

The Weather Bureau and the Army Engineers, thus far, have failed to appreciate the importance of the study of the water resources of the country, and it has thus been left to the United States Geological Survey, through its Hydrographic (now Water Resources) Branch to collect the much-needed information. In line with these remarks, the writer, in 1894, in referring to California conditions, said:*

"The great value of hydrometric data on rivers of the importance of the Sacramento and San Joaquin does not yet seem to be appreciated by the U. S. Army Engineers in charge of river improvements. Their attention has been directed solely to a maintenance of depth of water at the low stages, and the collection of data by them has not been extended beyond a study of the conditions at low-water stages; no high-water gaugings have been made, no river-rod records are being kept, yet one assistant could furnish, on the basis of frequent approximate gaugings of Sacramento River near Red Bluff, of Feather River near Oroville, and of American River near Folsom, with occasional measurements of the flow of smaller tributaries, a reliable exhibit of the volume of water entering Sacramento Valley.

"It may be claimed that the U. S. Engineer Corps is not interested in volumetric records at the high stages of the rivers; that their recommendations cannot extend beyond the conservation of the navigability of the low-water channels, and that all data pertaining to the river in flood should be collected by the authorities directly interested in land protection and reclamation, by the State, and by the land owners. But it must be evident that works to improve drainage should not be detrimental to commercial interests and vice versa. Questions of expediency in the matter of changes of alignment, cutting off bends, division and control of surplus waters, etc., which arise in connection with drainage problems, must be discussed in their bearings upon the navigability of the rivers, and, without data relating to the river in

* "Report of the Commissioner of Public Works to the Governor of California," 1895, p. 138.

Mr. Grunsky. all its stages, the U. S. Engineer Corps will be but poorly equipped to combat or acquiesce in the recommendations of engineers studying drainage problems."

The authors of the paper, too, are to be thanked for the contribution of this material to the Society, where its value will be enhanced by discussion.

The writer, while Assistant State Engineer of California, in charge of office computations, in the years following 1878, under William Ham. Hall, M. Am. Soc. C. E., who was then State Engineer, was entrusted with an analysis of flood conditions similar to that presented in this paper. At that time, however, the representatives of the Water Resources Branch of the U. S. Geological Survey were not in the field, and comparatively little was known of the water delivery of the individual streams into the Sacramento and San Joaquin Valleys. However, gaugings had been made and rating tables had been constructed for various points on Sacramento River. Some gaugings of tributaries had also been made, and it was possible, from the knowledge (though imperfect) of rain distribution and of run-off from certain areas, to approximate other run-off values and through these the stream flow for selected time periods. The attempt to combine these properly, with due allowance for the time at which each stream was at its maximum stage, the duration of the storm producing the high stage, the length of time that would elapse before the tributary produced a maximum effect upon the main stream, and the modifying effect of channel storage, led to a special study of flood-wave movement by the writer, and incidentally to the use (probably for the first time) of the mass-curve (though not under this name) which is now so generally made an aid in studying the influence of storage upon water yield.

The following discussion of the flood-wave problem, as then worked out, was published in 1895:*

"When a supply of water is received by a river from a tributary, in excess of the ordinary flow of the tributary, and the capacity of the river is sufficiently great to accommodate this addition to its volume of flow, then the river in the immediate vicinity of the tributary will rise, and the water thus elevated in a portion of the river channel will at once seek its level. The flow of the river and elevations of water surface will in sucessive time-periods be increased at all points below the source of supply; a flood-wave will travel down the river.

"The wave represents a rise of water above a normal condition, and if discharged into a long, narrow body of still water it would seek its level. The wave when launched upon a flowing stream has the same tendency of elongation, and the front of the wave therefore moves down stream at a speed greater than that of the current in the river before the flood-wave was created.

* "Report of the Commissioner of Public Works, California," 1895, p. 130.

"The form of the flood-wave is determined by time and elevation of water surface at any point. Mr. Grunsky.

"The discharge curve at any point corresponding to the time-period during which a flood-wave passes, is somewhat similar in form to the flood-wave as determined by elevation of the water surface, but the maximum discharge occurs before the crest of the wave reaches that point, except in cases when the maximum flow is long sustained.

"The velocity with which a flood-wave travels down stream is dependent:

"Upon the total amount of water which is supplied to produce the wave;

"Upon the rate of supply;

"Upon the character and dimensions of the waterway through which the wave travels;

"Upon the amount and velocity of the water already in the river.

"The flood-wave velocity varies therefore for each flood, for every river, and for the several portions of each river.

"The velocity of movement is not the same for the different portions of a flood-wave. In other words, the form of a flood-wave changes; it is elongated as it moves from point to point. It will extend its front more rapidly than its crest moves, while its extreme upper limit (which is marked by the falling of the water surface to the elevation which it would have held had there been no flood-wave) will not be much at variance with the velocity of the river current for that particular stage.

"Above the crest of the wave there must be a portion of the river channel partly filled with flood-wave water. The length of the up-stream portion of the wave is continually increasing in consequence of the more rapid advancement of the wave's crest than of its upper terminal point.

"The total volume of water represented by a flood-wave remains the same at every point passed, but the time consumed in passing will be greater for points near the mouth of the river than for points farther up stream, and the maximum discharge due to the wave must for this reason decrease as the mouth of a river is approached.

"If the dimensions of a river channel and its flow at some point, *A*, where all its waters are confined, are known, it may be required to determine what the greatest discharge at some point, *B*, farther down stream would be due to a flood-wave passing the point *A*, if no water were allowed to escape from the river channel between *A* and *B*.

"The discharge per unit of time at *B* must vary from that at *A*, because the time required by a flood-wave to pass *B* is greater than that required by the same wave to pass *A*. It follows directly that the mean discharge at *B* for the flood-wave period is less than at *A*, and supposing the forms of the wave at *A* and *B* to resemble each other, as will always be the case when *A* and *B* are not too far apart, the maximum discharge at *B* must also be less than the maximum discharge at *A*.

"Let q = amount of water in river channel between *A* and *B*, before the front of the flood-wave has reached *A*.

"Q', Q'', Q''', etc., = amount of water in river channel between *A* and *B* at the end of the time-periods t', t'', t''', etc.

Mr. Grunsky. "D', D'', D''', etc., = total amount of water which has passed A at the end of the time-periods t', t'', t''', etc.

"d', d'', d''', etc., = discharge at A in the time-periods t', t'', t''', etc.

"F', F'', F''', etc., = the total amount of water which has passed B at the end of the time-periods t', t'', t''', etc.

"f', f'', f''', etc., = discharge at B in the time-periods t', t'', t''', etc.

"Then:

$$d' = D'' - D'.$$
$$d'' = D''' - D''.$$
etc.

$$f' = F'' - F'.$$
$$f'' = F''' - F''.$$
etc.

$$D' = F' + (Q' - q).$$
$$D'' = F'' + (Q'' - q).$$
etc.

"The river will continue to rise at A until the crest of the wave passes A, and it then commences to fall, but ordinarily less rapidly than it rose. Consequently the value of Q increases until the crest of the wave is at some point between A and B.

"If the character of the river at B is similar to that at A, the rate of rise at B may be assumed greater than the rate of falling at A; and if the flood-wave is of sufficient extent to have its upper end above A when its crest is at B, the Q will attain its greatest value when the crest of the flood-wave is near B.

"Approximately, then, it may be assumed that the time at which Q is a maximum is the time at which the crest of the wave reaches B.

"The greatest discharge at B occurs just before the time the crest of the wave passes B, or very nearly at the same time when Q is a maximum. (This is only then the case when A and B are not too far apart, and the flood-wave is still passing A when its crest reaches B.)

"If the conditions are not as favorable as here supposed, then the time at which Q is a maximum, and the time at which the discharge at B is a maximum, must be separately approximated.

"In either case the discharge at B can be approximated with a considerable degree of accuracy by the following method." (See Fig. 3.)

"On a horizontal line time is scaled off. At the ends of the several time periods, t', t'', t''', etc., the values of D, total discharge past A, are plotted as ordinates, and establish a curve of total discharge at A.

"By subtracting from each of the ordinates of the 'D curve' the value $(Q_m - q)$, where Q_m represents the approximate value of the maximum channel storage due to the flood-wave between A and B, a parallel curve will result, the ordinates of which are $D - Q_m + q$.

"If the river before the flood-wave commenced to advance was in a normal condition, and the effect of tributaries between A and B be disregarded, then: the total flow (F) at B was the same as that (D) at A until the front of the flood-wave reached A, and if the successive values F', F'', etc., be platted as ordinates to establish a curve of total discharge at B, the curve thus determined will coincide with the 'D

curve' up to the time the discharge commenced to increase at A. The discharge curve for B can moreover be projected beyond that point for about the time required for the front of the wave to advance from A to B. Mr. Grunsky.

"When Q is at its maximum, then $F = D - Q_m + q$. In other words, the curve of total discharge at B, the 'F curve,' and the curve whose ordinates are $(D - Q_m + q)$, have one point in common.

"When finally the upper end of the wave has passed B, the two curves of total discharge at A and at B will again be coincident.

"The curve of total discharge at B thus has two points definitely fixed, its direction at these points is also known, and it must be tangent to another curve at some point between the former.

"Its form can therefore be approximated. The point which the 'F curve' has in common with the curve of the $(D - Q_m + q)$ can moreover be approximated by approximately determining the time at which Q is a maximum. * * * The approximation of the 'F curve' is greatly assisted by approximating water elevations for points intermediate between A and B, and therefrom calculating Q for as many points on the time scale as required.

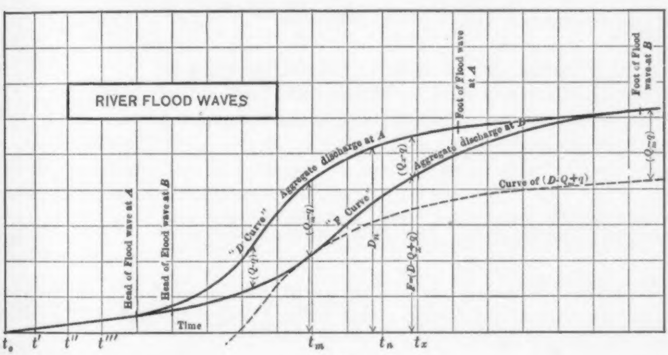


FIG. 3.

"Should the amount of water (Q) stored between A and B remain the same for some time after Q attains its greatest value, then the 'F curve' will for that time remain parallel to the 'D curve.' The greatest discharge at B will then be the same as the discharge at A at the time when Q becomes a maximum.

"As soon as the amount of water stored between A and B begins to decrease, the 'F curve' will begin to approach the 'D curve,' and the discharge at A will be less than at B.

"The discharge at A for a unit of time is the tangent of the 'D curve.'

"The discharge at B for a time unit is the tangent of the 'F curve.'

"From the foregoing it will appear without further demonstration that the greater the reservoir space which is provided in the river channel above any point, the less will be the maximum flow of the

Mr. Grunsky. river past that point, if the maximum supply from above does not continue beyond the time required to completely fill the reservoir space, and even then increased reservoir space above any point reduces the frequency of a maximum flow. It will always, therefore, be found advantageous when attempting to control floods by the construction of embankments along rivers, to place these far apart in the upper portions of the rivers, and to provide overflow or flood-basins, if the latter can be so arranged that they will not receive water at unnecessarily low stages, and are so located that a quick redelivery of their water into the river channel is possible."

The high-water stage of Sacramento River during a flood stage which occurred in March, 1879, based on data collected by Mr. Hall as State Engineer, may be analyzed as follows:

For the time period, March 4th to 20th, 1879, inclusive, the following mean values were determined:

	Cubic feet per second.	
Sacramento River discharge, Iron Cañon near Red Bluff (gauging station)	37 100	
From tributaries, Iron Cañon to Colusa	11 050	
		48 150
Channel storage, Iron Cañon to Colusa, elevation on March 4th to elevation on March 20th	990	
Sacramento River discharge at Colusa (gauging station)	45 780	
		46 770
Consequently, over-bank flow through breaks and crevasses above Colusa =		1 380

The over-bank flow above Colusa was, for the most part, on the east side, south of Chico Creek. This water passing through Butte Basin reached Sutter Basin.

	Cubic feet per second.	
Sacramento River discharge at Colusa (gauging records)		45 780
Sacramento River discharge at Knights Landing (gauging station)	19 000	
Channel storage, Colusa and Knights Landing	200	
		19 200
Consequently, over-bank flow between Colusa and Knights Landing = about		26 600

Of this over-bank discharge, only about 80 cu. ft. per sec. went to the west, all the remainder went over the east bank into Sutter Basin.

These figures checked well with the aggregate of estimates for each crevasse. Mr. Grunsky.

	Cubic feet per second	
Sacramento River at Knights Landing (gauging station)	19 000	
Feather River discharge into Sacramento River and into lower end of Sutter Basin	30 300	
		49 300
Discharge over-bank through crevasses in the right bank of Sacramento River between Knights Landing and Gray and Shaw's	11 150	
Sacramento River at Gray and Shaw's (gauging station) below Feather River	54 000	
Channel storage, between Knights Landing and Gray and Shaw's	120	
		65 270
Consequently, flow from Sutter Basin into Sacramento River = about		16 000

The water in Sutter Basin was about 5.75 ft. higher on March 20th than on March 4th. Its volume had increased by an amount equal to a mean flow of 20 400 cu. ft. per sec. The total quantity of water which this basin received in the 17 days, therefore, is (storage plus outflow) 20 400 + 16 000 = 36 400 cu. ft. per sec.

The flow of water from Butte Basin into Sutter Basin was estimated as follows:

	Cubic feet per second.	
Discharge of Butte, Table Mountain and other creeks into Butte Basin, and from that into Sutter Basin	810	
Over-bank flow from Sacramento River into Butte Basin, thence into Sutter Basin	1 370	
Total	2 180	
Sacramento River at Gray and Shaw's	54 000	
American River discharge into Sacramento River	13 000	
		67 000
Sacramento River at Freeport, 15 miles below American River (gauging station)	55 700	
Channel storage between Gray and Shaw's and Freeport	270	
		55 970

Mr. Grunsky.

		Cubic feet per second.
Excess of outflow from river, over inflow from American Basin		11 030
The over-bank flow into Yolo Basin at points between Gray and Shaw's and Freeport was estimated at		10 500
Consequently, American Basin, lying just above American River, received more water from the river than was delivered from the basin into the river, by about		530

Relating to American Basin, it was estimated:

	Cubic feet per second.	
American Basin received from Bear River across banks and from Feather River across banks	4 080	
American Basin received from creeks south of Bear River	1 060	
American Basin received from Sacramento River, over-bank, net	530	
		5 670
The water in American Basin on March 20th was 6 ft. higher than on March 4th. This increased storage is equivalent to		5 690

The discharge of Feather River for the 17 days is based on the following:

	Cubic feet per second.	
Feather River discharge at Burt's Ferry	21 660	
North and South Honcut Creeks	640	
Yuba River discharge	10 330	
Bear River discharge	2 210	
		34 850
Over-bank flow from Feather and Bear Rivers into American Basin	4 080	
Channel storage, Feather River below Burt's Ferry	470	
		4 550
Feather River discharge into Sacramento River		30 300

Had all the water been confined to the river channels above Gray and Shaw's (located a few miles below the mouth of Feather River) then the aggregate discharge of Sacramento River at that point would have been increased by the total over-bank flow above that point, and would have been decreased by the total flow into the river from Sutter Basin. In other words, the total increase would have been equal to the storage increase in Sutter Basin, 20 400 cu. ft. per sec., plus the flow over-bank from Feather and Bear Rivers into American Basin,

4 080 cu. ft. per sec., plus the flow over-bank from Sacramento River into Yolo Basin at points between Knight's Landing and Gray and Shaw's, 11 150 cu. ft. per sec., or, in the aggregate, 35 600 cu. ft. per sec. Mr. Grunsky.

The measured flow at Gray and Shaw's for the 17 days was 54 000 cu. ft. per sec. If all the water had been confined to the river channel, the total discharge there would have been 54 000 + 35 600 = 89 600 cu. ft. per sec., reduced by some small indeterminate amount, represented by channel storage, that would have resulted from the necessarily higher water stages.

The increased average flow of the river below the mouth of American River (Freeport gauging station) would have been the increase at Gray and Shaw's, 35 600 cu. ft. per sec., plus the contribution of American Basin creeks, 1 060 cu. ft. per sec., plus the outflow through crevasses in the river bank into Yolo and American Basins at points between Gray and Shaw's and Freeport, 11 030 cu. ft. per sec., or, in the aggregate, about 47 700 cu. ft. per sec. This increase, however, is again subject to some reduction on account of storage of water in the river channels resulting from the higher waters that, no doubt, would have resulted in a regulated river carrying the entire flood flow.

These amounts of increase in volume of flow are mean values for the entire period of 17 days. They do not represent the maximum momentary increase. It is safe, therefore, to say that the maximum would have been increased by at least these amounts and probably by considerably more. The mean value, for the purposes of an approximation, may be considered as about two-thirds of the maximum. On this assumption, the greatest discharge at Sacramento City (Freeport gauging station) would have been increased on March 13th from 71 000 to about 143 000 cu. ft. per sec.

Although, as already stated, the flow estimate of such streams as Yuba, Bear and American Rivers, Stony Creek and other streams was not based on gaugings during the flood conditions of March, 1879, their aggregate output could still be ascertained with a fair degree of approximation because nearly all their waters dropped into overflow-basins of known extent and of known water surface elevation. This fact, as shown in the foregoing analysis, proved a valuable aid in the discussion of what the river discharge would have been if all water had been confined to an adequate channel.

A flood discharge study for the 1879 conditions, similar to the foregoing, was made in 1879 and 1880 in the office of the State Engineer, and was supplemented by another estimate based on the mass-curve or flood-wave movement method, which has already been explained. Figures are not at command to give the basis of this second estimate in detail. It was necessary to assume a definite position of river levees on which the estimates of channel storage were then based. There-

Mr. Grunsky. upon, water elevations were approximated for the selected time period. Where data were lacking relating to the discharge of streams at the points where they entered the valley, discharge curves were controlled in shape by the known shape of discharge curves on other streams, and by a comparison of the run-off per square mile of drainage area. Reasonable time allowances were then made for the forward movement of the flood wave in each tributary, and flood-wave shapes were modified by making proper allowance for channel storage.

As a result of this second estimate, it was concluded that, during the flood period of March 4th to 20th, 1879:

The Feather River maximum delivery into Sacramento River would have occurred on March 6th, and would have been 57 400 cu. ft. per sec.

The American River maximum discharge would have occurred on March 6th, and would have been 37 000 cu. ft. per sec.

The Sacramento River maximum discharges would have been:

At Red Bluff................	133 000	cu. ft. per sec.
" Colusa	89 000	" " " "
" (above) Feather River.....	83 000	" " " "
" (above) American River...	136 000	" " " "
" Sacramento City	162 000	" " " "

A similar study was made later for the flood condition of January, 1894,* and the following probable maxima were determined:

Sacramento	River	at Red Bluff..............	152 000	cu. ft. per sec.
"	"	at Colusa	92 000	" " " "
"	"	at (above) Feather River..	87 000	" " " "
"	"	at (above) American River	136 000	" " " "
"	"	at Sacramento City	156 000	" " " "

The actual maximum discharge at Sacramento City in 1894 was only about 54 000 cu. ft. per sec. In January, 1894, the river rose 4 ft. higher at Red Bluff than during the flood stage of March, 1879. The river at Sacramento, in January, 1894, was 2 ft. lower than in March, 1879. At the earlier date the flood-basins were well filled with water; at the later date they were practically empty at the commencement of the rise and were therefore particularly efficient in retarding the passage of the flood wave down the valley.

No allowance for storage, outside of the present alignment of river levees, was made in arriving at the foregoing figures, although it is well known that it would be impossible to carry flood volumes between the present lines of levees.

*"Report of Consulting Engineers, Manson and Grunsky, to Commissioner of Public Works, California," "Report of Commissioner, 1895," p. 53.

Neither the flood conditions of 1879, which are the main basis of the foregoing discussion, nor those of 1894 were extreme conditions. Both floods were such as may occur frequently. That of 1879 was the result of rainfall conditions that have been described as follows:* Mr. Grunsky.

"After moderate rains in January, 1879, a general rain set in on February 7th. The storm continued six days, during which time 3.15 inches fell at Sacramento. After an intermission of a day, rain again fell during a three-day rainy period to the extent of .73 of an inch. The next two weeks were dry. It rained a little on March 2d, and a second general storm commenced on March 4th. On the 5th, the precipitation reached 1.97 inches at Sacramento, and during this storm, which continued to March 9th, the precipitation at Sacramento was 3.85 inches."

At the flood stage of 1907, according to the interpretation, by the authors, of the U. S. Geological Survey records at Red Bluff, the maximum discharge was 204 000 cu. ft. per sec., and there were five days in which the discharge exceeded 100 000 cu. ft. per sec. The mean flow for 11 days was 98 500 cu. ft. per sec., and for 17 days it probably exceeded 80 000 cu. ft. per sec. The Feather River at Oroville is shown to have been at its highest stage on the same day that Yuba and Bear Rivers were highest. The combined mean flow of these streams for 11 days was 112 800 cu. ft. per sec., and for a period of 17 days, probably about 86 000 cu. ft. per sec.

Comparing these values with those heretofore noted for the high water of 1879, it is found that the flow in 17 days at Red Bluff in 1907 exceeded that of 1879 by about 43 000 cu. ft. per sec. It was more than twice as great. For the same length of time, the discharge of Feather River and its two tributaries in 1907 exceeded the flow of 1879 by 52 000 cu. ft. per sec. In this case, also, the later run-off was more than twice as great as the earlier. The maximum at Red Bluff in 1907 was 54% greater than the maximum in 1879. This comparison is an indication that about twice the channel capacity of drainways would be required for a flood similar to that of 1907, as for one similar to that of 1879.

The water which would pass down the Sacramento Valley thus indicated for the latitude of Sacramento would be somewhat more than 320 000 cu. ft. per sec., not including Cache Creek or Puta Creek discharges. This amount is to be compared with 559 000 cu. ft. per sec. noted by the authors for Sacramento River below the mouth of American River, and 230 000 cu. ft. per sec. estimated by the Board of Engineers of 1904. As this comparison seemed to indicate an overestimate on the part of the authors, it led to a further examination of their figures. They have allowed in their estimate for the time that it takes flood waves to travel from point to point, but have not made

* "Report of Commissioner of Public Works, California," 1895, p. 57.

Mr. Grunsky. the proper allowance for the flood-wave shape. Thus, for example, the combined maxima of Feather, Yuba and Bear Rivers, 258 000 cu. ft. per sec., are assumed as being delivered into the Sacramento River at practically the same instant of time, without taking into account the reduction of these maxima that must result from flood-wave elongation. Allowance for such elongation would reduce the maximum delivery of these three streams into Sacramento River by about 20 000 cu. ft. per sec.

It can be shown in a similar way that the river reservoir space between Red Bluff and Feather River would have a like modifying effect upon the Sacramento River discharge, and that, consequently, the total discharge for which provision is to be made below the mouth of Feather River will be somewhat less than the 466 000 cu. ft. per sec. noted in the paper.

To get some idea of the maximum output of the Sacramento River below Cache Slough, under regulated conditions, that is to say, with enough waterway, natural and artificial, to carry the entire flood discharge, the following process is suggested:

Assume as an upper limit of the mean velocity at high-water stages, in the channels traversing the Sacramento Valley lengthwise 5 ft. per sec. and 7.5 ft. per sec. in the channels such as Feather, Yuba, Bear and American Rivers, which cross the direction of the valley. On this assumption, and a first approximation of maximum discharge, the cross-sectional area of the required channels can be determined. Let it then be further assumed that, of the cubical contents of the channels thus determined, variable amounts, ranging from four-fifths for the tributaries and up-river reaches to one-half for the lower stretches of the river, will be available as reservoir space to be filled during the passage of the flood. On this assumption, the total water stored in the channels at some time during the flood would have exceeded the channel contents at the beginning thereof by an amount equivalent to a flow of about 600 000 cu. ft. per sec. for one day.

Using the discharge figures given by the authors without correction, but apportioning the flow noted for valley and foot-hill areas to the several days of the flood period, it will be seen that the aggregate amount of water which entered Sacramento Valley was:

On	March	16	about	140 000	cu.	ft.	per	sec.
"	"	17	"	313 000	"	"	"	"
"	"	18	"	582 000	"	"	"	"
"	"	19	"	712 000	"	"	"	"
"	"	20	"	547 000	"	"	"	"
"	"	21	"	374 000	"	"	"	"

By the mass-curve method, already described, it can now be shown that the maximum discharge of the Sacramento River below its lowest

tributary would have been about 540 000 cu. ft. per sec., and that this would have occurred late on March 19th, or on March 20th. As the assumed mean velocities are probably in excess of those that would prevail in flood-water channels, it seems probable that the high-water channels would have to be correspondingly larger and that there would, in fact, be somewhat more channel storage effective in holding back flood-waters than above assumed. Mr. Grunsky.

It is believed that 540 000 cu. ft. per sec., as an approximation of the maximum discharge that would have resulted in Sacramento River below Cache Slough if water had been delivered into the valley as estimated by the authors, and if it had all been confined to adequate channels, is nearer the correct value than the 640 000 cu. ft. per sec. noted in the paper.

Not only is this probable, but it is thought likely that the authors have over-estimated the high-water discharges of most of the streams. The rating tables are based on gaugings made for the most part at relatively low stages, and are approximations for discharge at high stages, at which the disturbing effect of irregular channel is great. The writer believes, however, that the recorded discharge of Sacramento River through Iron Cañon, where the river lies in a beautiful straight reach and has a bottom permanent or nearly so, is not materially in error. But in the case of Feather River at Oroville, and the Yuba, Bear and American Rivers, the over-estimate is probable.

A run-off of 25 500 cu. ft. per sec. has been assumed by the authors for a 4-day period from the flat, arable, and for the most part cultivated, surface of Sacramento Valley. This is said to be 50% of the rain which fell in the valley from March 17th to 20th. It represents a run-off depth of about 0.85 in. The rain which caused it was therefore evidently taken at about 1.70 in. Now, it is well known that about 3 in. may fall in an ordinary rain storm, such as that of March 16th to 20th, extending, as did this, over several days, without producing any run-off from valley lands or from the adjoining low foot-hill region of the west. As an illustration of this point, the following experience of the writer may be noted. In April, 1896, while a study of a west side Sacramento Valley drainage project was being made, it commenced to rain and the storm proved to be general. Preparations were made to gauge the foot-hill streams, but they did not respond to the rain. Although nearly 4 in. of rain fell in four days on some portions of the region under consideration during a storm in which 2.84 in. fell in the same four days at Sacramento and 2.31 in. at Willows, the Coast Range foot-hill streams brought down no water. On this occasion, the foot-hill lands and the flat valley lands took up 3 in. of water without producing enough run-off to reach the trough of the valley. It is believed, in view of the foregoing, that the 25 500 cu. ft. per sec. are far in excess of the actual valley run-off. It is probable that one-tenth to one-fifth of this quantity would be nearer the truth.

Mr. Grunsky. Again, 76 000 cu. ft. per sec. are noted as the mean run-off for a 4-day period from low mountain and foot-hill areas, some on the east side of the valley (about 1 600 sq. miles), and some on the west (about 1 500 sq. miles). That on the west includes Red Bank, and Thomes Creeks; that on the east, Deer, Mill, Chico, Honcut and other streams which respond readily to winter storms. But much of the area is of the soil-covered foot-hill type, which absorbs moisture greedily, and for which the assumed run-off value for a 4-day period of 25 cu. ft. per sec. is undoubtedly too large. This run-off is equivalent in 4 days to a run-off depth of 3.72 in. To produce this in the course of a rainy season there would ordinarily be required about 19 in. of rain. It is not believed possible that in the low areas, where there was little or no snow, such an amount of water flowed to the streams.

In view, therefore, of the over-estimates of the run-off from valley and foot-hill areas, and the probable over-estimate of the discharge of the principal tributaries of the river, it may safely be assumed that the figures given by the authors in Table 24 are over-estimates, particularly those applying to the lower reaches of the river. The probability is that the figures for the actual water quantities to be provided for lie somewhere between those of the Board of Engineers of 1904 and those given by the authors, though probably nearer the latter.

The same criticism relating to over-estimates that has just been made with reference to the regulated flow of Sacramento River applies in the case of the San Joaquin. The rains of March, 1907, produced very little run-off from San Joaquin Valley lands, except on small areas near Stockton and possibly near Merced, where the soil is clayey. The total valley run-off was so small that it may well be neglected. It has been entered by the authors at 23 560 cu. ft. per sec. as a mean for a 4-day period. Neither did the foot-hill lands produce a run-off of 1.8 in., the amount assumed for which as an aggregate for a full ordinary rain year a rain of about 14 to 20 in. would be requisite. On the other hand, this flood was not typical of the occasional great floods in the San Joaquin Valley. All the streams southward from the Mokelumne River have at times been much higher. Such years as 1861-62 and 1867-68 would produce conditions very different from those which prevailed on the San Joaquin and its tributaries in 1907. There is no occasion, therefore, for attempting to apply here, also, a mass-curve analysis, though it is believed that if this were done the relative reduction of the total appearing for San Joaquin Basin in Table 21 would be much greater than the reduction above indicated as a probable correction to be applied to the total given for the Sacramento Basin.

The writer shares with the authors the opinion that it would be desirable to deal with the drainage and irrigation problems of the Sacramento Valley as a single project, and is convinced that water

storage in the mountain water-sheds, as pointed out in the paper, would be beneficial. The aggregate amount of storage that can be made effective in elongating flood waves, however, is relatively small. Under the high-water conditions as they prevailed at the critical period of March, 1907, the great reservoir on Pit River would have held back only about 200 000 acre-ft. Nearly 3 000 000 acre-ft. of its capacity, while useful in holding a water supply for irrigation, would have been useless in modifying the flood discharge of Sacramento River into the head of Sacramento Valley. In other respects, too, the full benefit of the reservoir capacities cannot be hoped for. Being intended to hold water in reserve for irrigation, they should be full or nearly full in spring, yet their maximum efficiency as modifiers of flood discharge depends upon their being empty or nearly so at the beginning of the critical period during a general rain storm, which may occur as late as April. Intelligent control of outflow from the reservoirs during the rainy season, therefore, is a requisite, if they are to be of material effect in reducing river discharge maxima. Mr. Grunsky.

The total amount of storage capacity in the reservoirs mentioned in Table 27 is about 4 800 000 acre-ft., of which, as explained, at least 3 000 000 acre-ft. would not have been effective, because the water was not available in sufficient quantity for storage in the Big Valley on Pit River. A part of the remaining 1 800 000 acre-ft. would likewise be unavailable for restraining flood waters in any month of March because, use for power and irrigation being assumed, the reservoirs might then already be at a half-full stage. The storage available, under such circumstances, to modify a March flood would be 900 000 acre-ft., equivalent to about one day's discharge from all sources into Sacramento Valley under such conditions of run-off as prevailed from March 17th to 20th, in 1907. The half-full stage in March is used here merely as an illustration of the principle involved. According to the quantity of snow on the ground, and other known conditions for each reservoir, there might be considerable departure from the half-full stage in the individual reservoirs.

H. M. Chittenden, M. Am. Soc. C. E. (by letter).—This paper has a particular interest to the writer because of the exhaustive study that he made of the flood problem of the Sacramento River in 1904, as a member of the Commission of Engineers appointed by the State of California to investigate that problem. One of the chief difficulties which the Commission encountered in its work was the lack of accurate data upon the flood discharge of the main river or its tributaries. The data which the authors have here presented in so complete and satisfactory a manner cannot fail to be of immense value in any future plans for the solution of that problem. Mr. Chittenden.

The writer notes with considerable satisfaction the records of the flood discharge of Stony, Cache and Puta Creeks. The only data

Mr. Chittenden. available at the time the Commission made its investigations gave such absurdly large figures for the discharge of these streams that the Commission deliberately rejected them, and arbitrarily assumed others which seemed to them to be nearer the facts. It now appears that even their estimates were too high; and it may be safely assumed that these streams, particularly Cache and Puta Creeks, will no longer be the bugbear which they were in previous investigations of this problem.

In regard to the maximum discharge of the Sacramento River in the recent flood, on the assumption of its being all confined to the channel, the writer is of the opinion that the figures given by the authors are somewhat excessive. As stated in their paper, no allowance was made for the attenuation of flood waves in their progress down stream, and the maximum discharge was taken as the arithmetical sum of the discharges of the tributaries, with a time allowance for their reaching certain specified points. While this assumption was probably more nearly correct in the late flood than it would have been in any other in the known history of the river, still the writer believes that it is not entirely correct, even in this case. The main stream receives no accession whatever in high water below Stony and Chico Creeks (at the upper end of the flood plain), except at the mouths of Feather and American Rivers and Cache Slough. If the trunk stream were treated so that it could carry a flood like that of 1907, its channel capacity would be so great that a large amount of storage would be absorbed in any considerable increase in gauge height, while, in the upper portion at least, there are so many obstructions to the regular flow that any computation of the rate of progress of a flood crest could hardly be depended upon as correct. In a short, sharp flood wave, like that of 1904, in which the discharge at Iron Canyon rose from 85 000 cu. ft. per sec. on February 15th to 185 000 next day and fell to 70 000 on the third day, there can be no question that the wave flattens out very rapidly on its way down stream; and, upon passing out into the bay, its maximum value would probably be not more than half of that at Iron Canyon, but it would be much longer in passing. Of course, this effect will be proportionately less pronounced, as a flood wave is of long duration, rising gradually to its crest and falling gradually therefrom; and this was exactly the situation in the flood of 1907. Still, the writer is inclined to think that, taking all things into consideration, there was some diminution of the volume of the flood wave at crest with the progress down stream, both in the stretch between Iron Canyon and the mouth of Feather River, and in that below the mouth of American Fork; but, with the utmost allowance for this effect, it is still apparent that the discharge vastly exceeded that of any other flood of which there is a definite record.

The writer is also inclined to think that the authors give greater

Mr. Chittenden.

weight to the effect of the proposed reservoirs in a flood like that of 1907 than they are justified in doing. Of the importance of reservoirs in storing the flood waters of streams, there can be no question. The value of this stored water is so great in many directions—for power, irrigation, and even navigation itself—that the utilization of all possible reservoir sites is justifiable from nearly every point of view. It is questionable, however, whether the creation of such reservoirs at a vast outlay is justifiable from the point of view of flood protection alone; and even if reservoirs were built exclusively for that purpose, it is doubtful if popular sentiment would not sooner or later compel their use for industrial purposes.

Now, when it comes to a combination of industrial use and flood protection, it will at once be found that the two purposes are very conflicting. For industrial use it is important that the reservoirs be full when the season of heavy precipitation terminates; for flood protection it is important that they be always kept empty until the danger of flood-producing rains is certainly passed. It will be understood, of course, that, in making this statement, the writer is speaking in general terms, and fully understands that a particular reservoir site may have special conditions which may modify it to some extent; but, as a broad generalization, unless reservoirs are given extraordinary capacity, it is not probable that they can be utilized to their full extent for protection against the larger floods. This is especially true when conditions prevail like those which caused the flood of 1907. The streams of the Sacramento Basin appear to have been practically in flood stage for a considerable period before the final storms of March 17th, 18th, and 19th arrived. Unless the various reservoirs (if built) were provided with enormous sluice-ways, capable of exhausting the storage nearly as fast as it came in previous to these dates, it would probably have been quite impossible to keep them from filling up.*

It must be remembered that a fatal obstacle to the perfect handling of any system of reservoirs, from the point of view of flood protection, is that Science has as yet furnished no means of predicting climatic conditions with any degree of certainty. It can never be foretold when or in what magnitude storms will come, and, therefore, the only safe plan to follow, if reservoirs are to be kept in use for purposes of preventing floods, is always to keep them as nearly empty as possible; but this, as previously stated, is contrary to the policy required if they are to be used for industrial purposes.

It is the writer's opinion that, of those reservoirs which have a sufficient drainage area above them to exercise an appreciable in-

*This, of course, refers to reservoirs of moderate capacity, and not to the immense ones like those on Pit River and at Clear Lake.

Wherever reservoirs exist of such vast capacity compared with the run-off from the water-shed above, it is clear that they are a perfect protection from floods in the valleys below them, so far as the run-off from above is concerned.

Mr. Chittenden. fluence upon a flood like that of 1907, not more than four could be depended upon to realize the authors' expectations, and that, of the remainder (on the Sacramento, Feather and Stony), not more than one-fourth to one-third of their actual capacity could be depended upon with certainty. Most of the flow of Puta Creek and about one-half of that of Cache Creek could be held back effectually.

While the writer is of the opinion that the authors' estimate of the value of the reservoirs in reducing a flood like that of 1907 is too great, still, as such floods are of extremely rare occurrence, probably not arriving more than once in a century, if even that often, their value in such floods is not a matter of so much importance as in the great floods, like that of 1904, which come every few years. They would be cut off enough from the peak of such floods to form an effective insurance in a scheme of improvement such as that recommended by the 1904 Commission; because, with the reduction in flood height which they would produce, the works proposed by the Commission would accomplish the desired flood control with almost certain safety from danger of serious damage.

As to the problem itself, of affording immunity from a flood like that of 1907, the writer quite agrees with the authors that the vast expense of such a work, on the lines proposed by the Commission, would probably exceed the resources of the State. It is doubtful if there is any practicable method by which such a deluge can be effectually controlled. The problem is certainly more intricate and difficult than that of the Mississippi, and, in proportion to the land area involved, is vastly more expensive. Its complicated and conflicting nature can be partially appreciated when the several ends which are sought to be obtained are considered, namely, flood control, navigation, and irrigation or other industrial use. It has already been pointed out how conflicting are the demands upon reservoirs for the two purposes of flood control and industrial use. The purposes of flood control and navigation are likewise, to some extent, conflicting. Navigation, in the low-water season, requires a concentration of the current into a small channel section; flood protection requires a very large section. To make the Sacramento River large enough to carry its great floods will necessitate an increase in the channel section throughout the whole extent of the flood plain, and a very large increase in certain portions. Take, for example, the reach between Feather River and Colusa—one of the most remarkable river channels in the world. Right in the middle course of a stream which comes down with an ordinary high flood stage of say 150 000 cu. ft. per sec., occurs this reach of 70 miles of firm and stable banks, excessive sinuosity, and a contracted section which can carry only about 30 000 cu. ft. per sec. within banks. This part of the river virtually says to that above: "I will receive and transmit so much water, and no more, and you will

have to dispose of the rest as best you can"; and the upper river promptly spills all the excess out into the basins on either side; and so the flow of this stretch of river varies from, say, 10 000 cu. ft. per sec. at low water to, say, 30 000 cu. ft. per sec. at high water, and is essentially the same as a navigable stream the year round. Why it does not fill up with sediment, considering its greatly reduced velocity from the reach above, it is difficult to understand; but the channel is nearly everywhere deep, the current gentle, and the stream an ideal one for navigation. It is also exceedingly picturesque, from a scenic point of view, with its regular curves, smooth water, and overhanging foliage. Mr. Chittenden.

All this will be utterly changed when the channel is developed to carry the great floods. The bends will be cut out, the channel distance shortened by about 23 miles, and the slope correspondingly increased. In time of flood a vast volume of water will flow rapidly along the enlarged channel, but in the dry season no more than at present. The result will be that the low-water flow, which now has a channel well adapted to its volume, will then find itself lost in a vast expanse of bed with the inevitable result of diminished depth, more shoals, and the usual defects of alluvial rivers.

No satisfactory answer occurs to this objection to a general scheme of flood control except that the importance of reclamation along this part of the river far outweighs that of navigation, and that the Government, which gave the State these lands on the condition that they should be reclaimed, cannot rightfully stand in the way of the necessary measures for their reclamation. The Sacramento and San Joaquin Valleys are the heart of California, and their development is of such vital interest to the State and the whole Pacific Coast that the problem of their reclamation must receive at least a partial solution.

The writer is of the opinion that the works proposed by the Commission of 1904, supplemented by the reservoirs which the authors propose, will solve the problem in any except extraordinary floods like that of 1907. It may probably be necessary for such floods to find relief in the basins for a portion of their volume, and if this can only be accomplished by suitable overflow weirs, so that the levees will not be destroyed, the very rare inundations which would result could probably be borne by the community without great hardship.

H. F. Labelle, M. Am. Soc. C. E. (by letter).—This paper, apart from giving valuable information regarding flood-producing storms, throws some light on the mooted question of the influence of elevation on rainfall. Mr. Labelle.

Some hydraulists are of the opinion that elevation does not always cause an increase in precipitation, especially in the United States. Fanning* has given consideration to this question, and shows that

* "A Practical Treatise on Hydraulic and Water Supply Engineering."

Mr. Labelle. rainfall generally decreases with the increase of distance from the ocean. He gives a table showing that in a series of rivers in different parts of America the rainfall decreases from their mouths to their sources. This is easily explained by the fact that the clouds, in passing over the land, are gradually depleted of their moisture, and that their capacity for producing rain gradually decreases, even with increase of elevation and consequent lower temperature.

When, however, the air is thoroughly saturated with moisture (and the writer will agree that this condition does not generally exist at long distances from the sea coast, at least in temperate regions), this air, coming in contact with mountain ridges and steep rises of ground, will deposit the moisture it contains at a more rapid rate as the elevation increases.

This influence of elevation on rainfall has been recognized for many years in India, and the following laws governing the same have been adopted:

1.—Rainfall increases with elevation, up to 5 000 ft.
2.—Rainfall decreases with elevation, above 5 000 ft.
3.—The increase of rainfall per 100 ft. rise is about 0.6 in.

This influence of elevation on precipitation is well illustrated by Buckley* who shows, on the map of India, the contour lines of rainfall and, consequently, its general distribution. By far the greater part of the rainfall of India is due to the western monsoon, which occurs between June and October. During that time enormous quantities of moisture are blown from the Indian Ocean in a northern and northeastern direction. These gyratory storms, called "taiphoons" by the Chinese, "baguios" by the Filipinos, and "cíclons" by the inhabitants of the islands of the Caribbean Sea, are sometimes more than 1 000 miles in diameter, and their western parts, after passing over the Arabian Sea, first impinge on the Western Ghauts, where the greatest rainfall occurs. After passing the summit of these hills, the clouds have lost their density, and the incidence of annual rainfall decreases, almost suddenly, from 100 to 25 in. Gradually, however, as a higher latitude is reached, the rainfall increases again, this increase cumulating in a maximum just north of the Central Provinces, after which it decreases until the foot-hills of the Himalayas are reached. The eastern parts of these cyclonic storms cross the Bay of Bengal and continue to gather strength and moisture until they reach the latitude of Calcutta. Owing to this fact, the precipitation is greater in the eastern than in the western part of India, and the average rainfall goes on increasing from 75 in. on the coast to 150 in. in the mountains of Assam. After passing these mountains, it again drops to 75 in., and, finally, it goes up to 100 in. and more, as the Himalayas are reached.

* "The Irrigation Works of India," p. 6.

Mr. Labelle.

Above 5 000 ft. the rainfall decreases, and before ascending to the top of the mountains, it has fallen to 30 in. It is seen, therefore, that, in India, there is always enough moisture left in the air during the western monsoon, from which India obtains most of its rainfall, to produce copious precipitation when the air currents strike the cooler mountainous regions.

In the authors' Table 2, an opportunity is given to test these laws relating to the influence of elevation on rainfall. A perusal of this table will show clearly that the first two laws mentioned are abundantly corroborated, and the few discordant figures found therein are not sufficient in number to change the general trend of the table, which shows that, for the type of storm and water-shed under consideration, the rainfall increases with the altitude to about 5 000 ft., and decreases for higher altitudes. In order to investigate the increase per 100 ft., the writer has prepared Table 28 from Table 2.

Table 28 shows the increase of rainfall between the lowest elevations and each successive higher elevation.

TABLE 28.—Increase of Rainfall per 100-Ft. Rise in Altitude, for Different Heights.

Basins.	Elevation, in Feet.					
	500-1 000	1 000-2 000	2 000-3 000	3 000-4 000	4 000-5 000	Mean.
	inches.	inches.	inches.	inches.	inches.	inches.
Sacramento	0.04	1.06	0.75	0.62	....	0.62
Feather	....	....	1.17	0.73	0.63	0.84
Yuba	....	0.02	0.94	0.56	0.62	0.54
Bear	....	....	0.74	0.37	0.37	0.49
American	0.43	0.37	0.50	0.50	0.55	0.47
Mokelumne	1.95	0.52	0.51	0.50	....	0.87
Calaveras	0.16	....	....	....	....	0.16
Stanislaus	2.22	....	....	....	....	2.22
Tuolumne	1.73	1.08	0.71	0.35	0.53	0.88
Merced	....	....	....	0.50	0.53	0.52
San Joaquin	....	....	0.45	....	....	0.45
Stoney Cr	0.45	0.93	....	....	....	0.69
Cache Cr	....	0.26	....	....	....	0.26
Puta Cr	....	0.06	0.84	....	....	0.45
Average	1.00	0.54	0.77	0.52	0.54	0.70

Table 28 shows that:

1.—The increase of rainfall from 0 to 3 000 ft. altitude is about 0.8 in. per 100-ft. rise;

2.—The increase of rainfall from 3 000 to 5 000 ft. altitude is about 0.5 in. per 100-ft. rise;

3.—The increase of rainfall for the whole area is about 0.7 in. per 100-ft. rise;

4.—The discordance between increase of rainfall, in the different sub-water-sheds, decreases with the elevation, the increase between 3 000 and 5 000 ft. being fairly constant for all subdivisions of the basin under consideration.

Mr. Labelle. Returning to Table 2: If the drop of temperature is computed for each 100-ft. rise of altitude, it is found that:

1.—The temperature drops with the altitude, in varying amounts per 100 ft. (0.05° to 0.46°).

2.—The quantity of decrease of temperature per 100 ft. increases with the altitude to about 2 500 ft. and is then constant to 5 000 ft.

3.—The mean decrease for all sub-water-sheds is as follows for each successive elevation:

Elevation above sea, in feet..	750	1 500	2 500	3 500	4 500	Mean.
Average temperature drop per 100-ft. rise..............	0.18°	0.17°	0.34°	0.33°	0.32°	0.27°

Apart from the fact that the drop in temperature produces an increase in rainfall, the writer cannot see any more intimate relation between the given figures of temperature and rainfall. The quantity of precipitation is affected by other factors than temperature, and among these, topography is, no doubt, one of the most important.

To exemplify this influence of topography, in combination with altitude, on the amount of rainfall, the writer will give as an extreme case, the rainfall at San Antonio, a point in the highlands of Central Luzon. The prevailing winds in Luzon, starting from the north at the beginning of the year, make a complete circle, in twelve months, in the direction of the hands of a clock. During the first half of the year the prevailing winds blow from the east or from the Pacific Ocean; this is the period of the eastern monsoon. During the second half of the year the prevailing winds are from the west or southwest, or from the China Sea; this is the period of the western monsoon. Therefore, the highlands of Central Luzon receive abundant rainfall from both sides of the island. Starting from Manila on the west coast and going in a southeast direction, there are two weather stations, the first at San Antonio, about 1 000 ft. above sea level, and the second at Atimonan, on the east coast. For the water year beginning July 12th, 1904, the rainfall at the three stations was as follows: Manila, 96.5 in.; San Antonio, 119.7 in.; and Atimonan, 96.7 in. During the year San Antonio received 23 in. more rainfall than the coast stations, or at the rate of 2.3 in. per 100 ft. rise in altitude, owing to its location being exposed to both monsoons. If this increase is attributed equally to each monsoon—and the conditions of the prevailing winds seem to warrant this assumption—there is an increase of 11.5 in. over each coast station, or 1.15 in. per 100 ft. rise; this is only 15% more than the average for the same elevation given in Table 28.

From the foregoing facts it is seen that, according to conditions, rainfall may or may not be influenced by elevation, and that this in-

fluence, depending on topography, etc., may vary within wide limits. This question, however, cannot be settled by isolated cases; it is only by using long-time rainfall records from a great variety of watersheds, and by subdividing these into several classes, that logical conclusions can be reached respecting this matter. Mr. Labelle.

In Table 21 the authors estimate the run-off from unmetered mountains and foot-hills to be 50% of the precipitation. Owing to the fact that the period, March 18th to 21st, was preceded by heavy rainfall, and that, besides, the mountain slopes and foot-hills are described in the paper as steep, barren and impervious, the writer believes that the run-off should have been taken at about 80% of the rainfall, but no doubt there were good and substantial reasons for adopting the lower figure.

Luther Wagoner, M. Am. Soc. C. E. (by letter).—This paper is an extremely valuable contribution, and presents the main facts of the flood with great clearness. The opening statement, that it was one of the most destructive floods that has ever occurred in California, while probably correct in a financial sense (and due to the fact that there was more property to be damaged than at previous floods), implies that it was about the greatest flood on record. The authors say: Mr. Wagoner.

"It is doubtful if any combination of causes or conditions will ever produce a larger rate of delivery of water to this valley for a 4-day period than occurred during the flood of March, 1907."

The writer believes that it would be unsafe to accept this statement as a basis for planning reclamation and flood prevention, unless it is qualified by a large factor of safety. It is generally believed that the flood of 1862 was greater in volume of water discharged into the basins and bay. In 1890 the writer, while engaged upon plans for the La Grange Dam on the Tuolumne River, found a well-preserved record of the 1862 flood near the present dam and 70 ft. above the bed of the stream. The record was in the shape of rounded pieces of wood and bark of fir, pine, tamarack, and juniper, showing that these pieces came from the higher regions. They were found in a talus of loose rock, and were doubtless carried into the void spaces by eddies and lodged there. Almost opposite, and across the river, a similar deposit was found, and at almost the same level. This led to a search along the river gorge above, where several similar records were found. Levels were taken, and connected with several cross-sections, and these, combined with the known high water at La Grange, about 1 mile below the dam, where the channel is wider and more regular, led to the conclusion that the maximum discharge was 130 000 cu. ft. per sec. This was based on the slope and Kutter's formula ($n = 0.040$), and the dam was planned to be able to discharge that volume over it; this corresponds to a run-off of 86.7 cu. ft. per sec. per sq. mile.

Mr. Wagoner. In 1895 the writer found a similar record on the middle fork of the American River near Volcanoville, from which, by the same methods, a run-off greater than 100 cu. ft. per sec. per sq. mile was deduced. (The original notes of both the foregoing records were destroyed in the San Francisco fire, two years ago.) Estimates have appeared in print in which the flood flow was given for the whole basin of the American River at 250 000 cu. ft. per sec., and even more. The record given above, applied to the whole water-shed above Fair Oaks, would give a discharge of about double the authors' 93 000 cu. ft. per sec.

While it may be true that a flow of 782 000 cu. ft. per sec. for 4 days may not be exceeded, there are two points to be considered. The flow from the San Joaquin region might occur as in 1862, and in combination with a 1907 flood on the Sacramento, in which case the quantity would be greatly exceeded. Again, suppose the rivers were leveed in accordance with the plans of the Engineering Commission of 1904, it would not require a 4 days' sustained flood to overtop the levees, and the probabilities are always in favor of the shorter but perhaps more intense run-off.

There is an average difference of a month in the melting of the snow upon the Columbia and Snake water-sheds, yet in 1900 it melted on each at the same time, with the result of backing up the Willamette and flooding Portland to a depth of several feet. This flood was sustained for more than two weeks.

The writer concurs in the conclusions of the authors, that relief from damage by floods must be sought in storage to relieve the peak of the discharge. Storage of débris is equally important, if the bed of the stream and navigation interests are to be preserved, and future studies should be upon the lines of effecting both water and débris storage.

It is the writer's belief that this can best be accomplished by loose-rock dams, backed with earth and waste on the up-stream face and with the down-stream face secured to the mass by suitable anchors, so as to allow the passage of floods over the crest of the unfinished dam during construction. When completed there would be an ample spillway around the dam, so that it could never be overtopped. Such dams would have to be of great height, from 400 to 500 ft., or even greater, because they would usually be located in gorges, and it might require from 300 to 400 ft. of permanent elevation to create a sufficient reservoir area, after which the increase of storage would be rapid. The only serious objection to such a type is the cost, but it can be shown that in the end it would be economical, because the desired regulation could thus be obtained (and, incidentally, the storage of débris), as well as power and irrigation.

It is beginning to be recognized that the proper treatment of this subject is a serious matter, and that the cost may reach $100 000 000

or more. From the analysis presented in the paper, it appears that there should be a storage of about 3 000 000 acre-ft. in order to give the required relief on the Sacramento water-shed alone. Such storage could be valued as follows: (*a*) Relief of peak load and flood prevention; (*b*) storage of débris; (*c*) preservation of the channel of the river; (*d*) irrigation; and (*e*) power. When all these possible uses are admitted and properly valued, it can readily be seen that a high cost per acre-foot of storage is permissible. **Mr. Wagoner.**

H. H. Wadsworth, M. Am. Soc. C. E. (by letter).—Although disastrous to the agricultural interests of so large an area, and to all the transportation lines of the valley, in some respects it may be said that this flood occurred at a very opportune time. The reclamation of the overflowed lands of the Sacramento and San Joaquin Valley has become a subject of vital importance to the future development of California, and the newly-awakened interest in the improvement of the navigation of these streams, and the movement toward the proper correlation of the various interests affected by the flow of the streams, from their sources in the mountains to their discharge through the Golden Gate, make the new standard set by this flood of great importance. **Mr. Wadsworth.**

The country is to be congratulated that the Geological Survey had a sufficient number of gauging stations established to obtain so many data in regard to the flood; and the authors are to be complimented on having presented them so promptly in such shape as to be of practical use, and show the magnitude of the problems involved.

A press of work, in connection with surveys looking to the regulation and improvement of these rivers, prevents the writer from discussing at this time more than one or two of several points or questions which have occurred to him, and these only very briefly.

As stated by the authors, previous estimates of flood flow of the Sacramento River have been greatly exceeded during this flood. In the case of the Yuba River, the maximum flow which has been assumed since the inception of the project for restraining débris in the bed of the stream, and which was used in designing the several works forming part of the project, is 125 000 cu. ft. per sec., or 25% in excess of the maximum observed flow at the gauging station of the Geological Survey. Before the failure of the dam, known as "The Barrier," this structure served as a weir for measuring the flow of the river, though its coefficient was very uncertain after it became backed up with tailings to its crest. The high-water marks left by the river during the night when the failure occurred indicated that the assumed maximum was nearly if not quite reached. Based on observations and estimates of flow at this point and at a few points on mountain streams carrying the drainage from comparatively small areas, Table 29 was prepared by the writer as a guide in determining the required capacities of spill-

Mr. Wadsworth. ways or canals to carry flood water around or away from dams erected for the storage of mining tailings.

TABLE 29.—Assumed Maximum Run-off, Sierra Nevada Streams, Based on the Flood of March, 1907.

Drainage Area, in Square Miles.	Maximum Run-off, in Cubic Feet per Second.		Drainage Area, in Square Miles.	Maximum Run-off, in Cubic Feet per Second.	
	Areas below elevation 4 000 ft.	Areas above elevation 4 000 ft.		Areas below elevation 4 000 ft.	Areas above elevation 4 000 ft.
1	544	408	40	8 650	6 490
2	915	686	50	10 200	7 670
3	1 240	930	60	11 700	8 800
4	1 540	1 150	70	13 200	9 860
5	1 820	1 360	80	14 500	10 900
6	2 090	1 560	90	15 900	11 900
7	2 340	1 760	100	17 200	12 900
8	2 590	1 940	200	28 900	
9	2 830	2 120	300	39 200	
10	3 060	2 290	400	48 700	
20	5 140	3 860	500	57 500	
30	6 970	5 230	1 000	96 800	

Table 29 is applicable to drainage areas of the character of the Bear and Yuba Rivers, and of such portions of other drainage areas as do not contain broad flat valleys. Of course, it would not apply to portions of the Feather River drainage area, containing the Sierra, American or Indian Valleys, or Big Meadows.

Considering for a moment the effect of mining débris on floods, it should be borne in mind that only on the Yuba River is it likely that the flood plane will continue to rise materially owing to this cause. The beds of the American, Bear and Feather Rivers (Feather above the mouth of the Yuba) have reached as near a state of equilibrium as is common for streams flowing through alluvial formations. The curtailment and regulation of hydraulic mining has largely stopped the accumulation of tailings in the torrential tributaries of the Yuba, and many of the cañons of these tributaries, which a few years ago were filled with tailings to a depth of from 20 to 60 ft., have been scoured out to bed-rock. Below the gauging station of the Geological Survey, at The Narrows on the Yuba River, there is a vast deposit of mining tailings standing on slopes which succeeding floods will continue to readjust; but any further extensive rise of the flood plane at Marysville does not seem probable.

In connection with the subject of partial control of the flood flow of the Sacramento River by reservoirs, it is interesting to note that the total capacity of five flood basins of the Sacramento Valley, as computed by the authors, amounts to 3 731 000 acre-ft., and that the effect of these basins was to delay the arrival of the flood crest at Rio Vista

about 4 days. Had there been levees sufficient to confine the river to the channels, how much higher stage would have been reached at this point? Mr. Wadsworth.

Of the storage reservoirs located and surveyed by the United States Reclamation Service, the total estimated capacity is 4 817 000 acre-ft., or about 1 000 000 acre-ft. in excess of that of all the Sacramento Valley flood basins. The latter are much more likely to be empty, or at least to have considerable capacity for the storage of flood waters when a flood occurs, than are the former, since, owing to the uncertainty of further large run-off before the dry season, it would jeopardize the agricultural interests dependent upon irrigation to leave these reservoirs nearly empty until after the middle of March. The worst flood of the season is likely to occur after that time, as was the case with the flood of 1907. On the other hand, no flood may occur, as has been the case during the season of 1908.

In whatever way the control of ordinary floods may be effected, it will very likely be found that:

"The task of rectification and enlargement of channel necessary to pass such floods as that of March, 1907, is so great as to make it economically impossible."

George L. Dillman, M. Am. Soc. C. E. (by letter).—The conclusion expressed in this paper is that mountain storage will be the ultimate solution of the flood problem of the Sacramento Valley. With this, the writer would take issue. He here states that the storage outlined is impracticable; also, that, if accomplished, it would prove inefficient, insufficient, precarious, and temporary. These hard names do not all apply to each case, but some of them apply to each case, and all to a few of them. Mr. Dillman.

The reservoirs mentioned, except Big Meadows and Clear Lake (which will be built by private corporations for power purposes), should never be built for flood regulation. The reasons are various. Some of these reservoirs are located where their water capacity would diminish rapidly by filling with débris, ultimately reaching zero. This applies specially to the Stony and Putah Creek Reservoirs, and also to Iron Cañon.

Some of these proposed reservoirs would flood lands which are too valuable to buy for such purposes. This is specially true of Indian Valley, where the lands which would be flooded are the finest kind of dairy lands, and include three small towns, with improvements which would make the cost great. From an irrigation standpoint, this storage is not needed, and would destroy more agricultural land than it would reclaim, less the cost of reclamation.

The Big Valley Reservoir—two-thirds of all the storage proposed—would be fed by an arid country. No doubt, its capacity compared with its cost is quite favorable, but some water should be allowed to

Mr. Dillman. pass to users between Bieber and Fall River. The evaporation would be large, and it would probably take several years to fill it. The authors' figures show that the 4 days' abnormal flood would fill 6% of it, if all was stored. During part of every year, the inflow would not equal the evaporation.

There is no reason, from an irrigation standpoint, for making the Feather River storages. Feather River is not much used for irrigation, by reason of expensive diversion. At low stages there is water without storage for any who will divert it; and there are no adverse claimants for it.

These storages are precarious. The high-water mark for years on the East Branch of the Feather was made by the failure of a small dam above Indian Valley. The flood of 1907 caused the failure of a dam on the Yuba, when the waters flooded large areas, some going south and breaking across Bear River, finally reaching American Basin. The possible damage by the failure of a reservoir dam on top of a flood can hardly be estimated.

The paper assumes the removal of water from the crest of a flood. This would hardly be the fact. This flood's crest came after weeks of flood, and it is fully supposable that the reservoirs would have been filled and held full long before the time had arrived for such beneficial effects. The location of these reservoirs averages more than 100 miles from the seat of damage. What man is wise enough, during a period of such storm and flood, to say when the psychological moment arrives for closing gates and making storage?

Taking from Table 27 the volume available for storage, or the capacity of the reservoirs, the total storage for this flood would have been 931 300 acre-ft. The side basin capacity is given as 3 775 000 acre-ft., or more than four times the mountain storage. This basin storage, like the mountain storage, would be largely made prior to the crest of the flood, but there is another factor. At about the time when the crest of a flood occurs, side levees break. In 1904, the Edwards break, on the east bank of the Sacramento, and in 1907, the Kripp break, on the west bank, gave great relief, as far as flood heights on the lower river go. The first poured into the Sacramento Basin and discharged into the San Joaquin through the Mokelumne. The second poured into the Yolo Basin and discharged into the Sacramento through Cache Slough. These side basins are not only storage reservoirs, but are great by-passes, through which flows a volume sometimes greater than that of the main stream. At flood times Yolo Basin is a stream 10 miles wide, flowing rapidly. The basin relief is from flow more than storage. It is at hand, and acts automatically at the right time.

Years ago, an avalanche of débris from hydraulic mining started toward the valley. Mining has stopped, but the débris is still coming.

Diversion of waters, whether natural or assisted, lessens the current, increases deposits of débris, and raises the beds of streams. This silt problem cannot be divorced from the flood problem. As a rule, the stream beds are rising, so that the same volume of water reaches a height increasing with time. Mr. Dillman.

From all the foregoing it would seem that mountain storage is not advisable for flood control. The valley problem is a complicated one, including the protection of lands, the navigation of streams, the drainage of flood water, and the care of the débris. Different solutions have been proposed.

The State Board of Works, about 1890, proposed an elaborate system of by-passes through the valley. This never met general approval. Such an installation would increase silt deposit in the streams, raise the flood and ground-water planes, and increase the area affected by floods. The ultimate effects would be detrimental to agriculture, navigation, and flood conditions.

In 1905, a Board of Engineers recommended a plan based on concentration, instead of diversion, of waters. The plan met general approval as to method. The cost, however, was great, though the agricultural land reclaimed would have been worth several times the outlay. The interests are so many and so divided, each wanting to benefit out of proportion to the outlay, that no action has resulted.

It seemed once that the débris problem might be divorced from the others by a series of débris barrier dams. The failure of the first one, on the Yuba, settled it otherwise.

The mountain storage solution has been brought up from time to time, but has never stood close analysis. To obviate the floods by preventing them, sounds all right until the illusion is dispelled by the cost, and the amount of storage is compared with the flood volume. The valley must take care of the floods. They will come. Mountain storage increases the risk, with very small compensation in possible results.

A rational solution lies in adequate waterways, made and maintained by a combination of dredging and levees. If plans to this end could be adopted, and all future work be obliged to conform to them, the end would be reached in time, without enormous initial outlay. As the plan was executed, cross and other auxiliary levees could be gradually abandoned as they became useless. River levees could be strengthened annually by dredging, as reclamation decreased the basin effect on floods. The levees would protect agriculture and confine the waters. Dredging would lessen the necessary height of levees, furnish material for them, and assist in the débris problem. The necessary height of levees, and the depths and widths of channels are the main questions. Rectification of alignment and foundations are important auxiliaries. As far as the Sacramento is concerned, débris has largely

Mr. Dillman. solved the foundation problem. The unstable peat bogs have been silted up, and now sustain enormous levees on Grand Island and elsewhere. The San Joaquin foundations are more serious problems, because the silt is deposited long before the waters reach the delta.

Mr. Duryea. EDWIN DURYEA, JR., M. AM. SOC. C. E. (by letter).—Messrs. Clapp, Murphy, and Martin are to be congratulated on their valuable paper, and especially is Mr. Clapp to be commended for so promptly seizing the opportunity to measure these floods. The modifications necessary in his field arrangements in order to enable him to cover so fully, before it was too late, the many streams observed, must have required very energetic and quick action on his part. The authors should be thanked also for their prompt presentation of the valuable information secured, which would not have appeared until at least a year later if it had been reserved for the Government publications; and even then probably it would not have been in such complete form as in this paper, and with such practical conclusions as are shown in Table 24 and in Fig. 2.

Discussions of papers generally take the form of criticisms of the authors' statements, but the writer has only one criticism to make on this paper, and that so unimportant and so little related to the subject of the paper that the only excuse for giving it is that otherwise a misstatement might appear uncorrected in the *Transactions*. The authors state that "the warm Japanese ocean currents, which bathe about 1 000 miles of the coast line, serve to equalize the temperature" of California. From recollections of a conversation with Professor Alexander G. McAdie, of the United States Weather Bureau, about five years ago, the writer doubted the correctness of this statement; and, on inquiry of Professor McAdie, received the following reply:

"You are quite right in your criticism of the statement made, that 'the warm Japanese ocean currents, which bathe about 1 000 miles of coast line, serve to equalize the temperature.' It is not even known that there is any such current on the Pacific Coast. In fact, the record of drift of wrecks shows that, for the most part, they drift northward, which would be exactly opposite to the assumed direction of the so-called Japanese Current. Very little is known of the current, even on the Japanese side, which is the only place where it is at all marked. So far as affecting the climate of California, the influence of the Japan Current is 'nil.' There are other and sufficient reasons why the western coasts of continents are warmer than the eastern coasts. Please see Bulletin L 'Climatology of California' page 7, for the great factors controlling Pacific Coast climate."

On page 7 of the "Climatology of California" (also written by Professor McAdie), it is stated as follows:

"The climate of California may be said to be controlled by four great factors. These are:

"1. The movements of the great continental and oceanic pressure areas—the so-called permanent 'highs' and 'lows'. * * * *

"2. The prevailing drift of the atmosphere in temperate latitudes from west to east; Mr. Duryea.

"3. The proximity of the Pacific Ocean with a mean annual temperature near the coast line of about 13° C. (55° F.), a great natural conservator of heat, and to which is chiefly due the moderate range of temperature along the coast from San Diego even to Tatoosh Island; and

"4. The exceedingly diversified topography of the country for a distance of 200 miles from the coast inland."

Again, on p. 15 of "Climatology":

"The active factors (in modifying the climate of the Pacific Coast) are the prevailing easterly drift of the atmosphere and the proximity of the mass of water, a great natural conservator of heat. * * * Too much emphasis can not be laid upon the effect of these two factors, the easterly drift of the air and the proximity of the ocean in modifying climate. It is probable that if * * * the general movement of the air in these latitudes (were) from east to west * * * the Pacific Coast might then have a rigorous climate."

The subject matter of the paper necessarily does not admit of much closely related discussion, as the only aim is the presentation of new data, bearing especially on the design of projects for flood control.

The necessity for the control of floods in the Sacramento and San Joaquin Rivers arises from the great losses to the ranches, leveed islands, cities, and railroads along the lower rivers, caused annually or nearly so by flood overflows; and from their injury to the navigable channels of the rivers. The uninterrupted navigation of these two rivers is very desirable, but the prevention of the frequent large property losses on the bottom-lands seems much the more important phase of the situation.

The great importance of the information collected and presented by the authors is best seen by a percentage comparison of the quantities given in Table 24, as shown in Table 30.

TABLE 30.—Maximum Rates of Flood Flow, Sacramento River.

Below mouth of:	Assumed by 1904 Engineering Commission. Cubic feet per second.	Computed from flood of March, 1907. Cubic feet per second.
Stony Creek	180 000 = 100%	261 000 = 145%
Feather River	190 000 = 100%	466 000 = 245%
American River	230 000 = 100%	559 000 = 243%
Cache Slough	250 000 = 100%	640 000 = 256%

The computed values of the flood of 1907, therefore, are about two and one-half times as great, below the mouth of Feather River, as the values assumed as maximum rates by an able Engineering Com-

Mr. Duryea. mission after the floods of 1904; and the importance of the 1907 rates, as governing the design of any engineering project to control the floods, is self-evident.

While the flood of March, 1907, is spoken of by the authors as the highest ever known in the Sacramento River, and is presumably the highest ever measured, there seem to be several reasons why it is imprudent to accept this view, at least to the extent of adopting the "computed" rates of Tables 30 and 24, unmodified, as a guide to the design of controlling works. In the first place, the authors state: "In all cases it is believed that the estimates are quite conservative, and rather inclined to be too low than too high"; hence, in adopting safe rates for this flood, it would be necessary to increase the "computed" values somewhat. Next, it is far from certain that the flood of March, 1907, has never been exceeded. The earliest flood measurements of the Sacramento and other Sierra streams known to the writer are those made by William Hammond Hall, M. Am. Soc. C. E., State Engineer, in 1879, and there seems to be no definite knowledge of their flood conditions before that time. Even since 1879, the flood records are not complete, no measurements having been taken during the ten years from 1885 to 1894, inclusive. Therefore, the only definite knowledge of Sierra floods generally available is for the six years from 1879 to 1884, and the thirteen years from 1895 to 1907, during which periods the Sacramento and some of the other Sierra rivers were measured.

The beds of Sierra rivers have been rising for many years past, however, and great floods in the earlier years must have caused much less damage than equal floods at present. Hydraulic mining, in progress from about 1852 to 1880, lodged immense volumes of débris in the upper portions of the rivers, and this débris still continues to be washed down by the annual floods into the lower portions. It has been stated:*

"The low-water plane of the Yuba River at Marysville was raised 15 ft. between the years 1849 and 1881. * * * The depth of fill (débris) varies (1905) from about 7½ ft. at Marysville to 26 ft. at Daguerre Point and 84 ft. at Smartsville. A short distance east from Marysville the bed of the river (Yuba) is now 13 ft. above the level of the surrounding farms. It is plain that any accident to the levees near Marysville would mean a disaster to the town.

"The quantity of material lodged in the river due to mining has been variously estimated at from 71 000 000 to 700 000 000 cu. yd., but it seems safe to say that there are now (1905) upwards of 333 000 000 cu. yd. in the bed of the lower Yuba. * * *

"Surveys recently made indicate that between 1899 and 1904 an addition of more than 15 000 000 cu. yd. (of débris) was lodged in the bed of the Yuba River from these old (and higher up) deposits. * * *

* "The Control of Hydraulic Mining in California by the Federal Government," by William W. Harts, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LVII, pp. 10 and 11.

Mr. Duryea.

"The Feather, the American, Cosumnes, Calaveras, Mokelumne and other tributaries of the Sacramento are all, in greater or less degree, affected in the same way.

"The Sacramento River itself shows unmistakable signs of considerable fill, largely due to mining detritus. Since 1849 the low-water plane at Sacramento has been raised about 7½ ft., causing a reduced carrying capacity for its flood waters, and requiring property owners to build levees to protect themselves from floods. The available depth for navigation, however, has remained about the same. The quantity of débris in the river (Sacramento) is estimated at 108 000 000 cu. yd."*

It is evident, therefore, that, even if a larger flood than that of March, 1907, occurred previous to 1879, the less obstructed stream channels of that early period would have passed the flood with less overflow than now; and, because of the smaller property values then subject to damage from overflows, even an equal overflow would have caused less damage; and a greater flood rate than that of March, 1907, may have occurred, even between 1849 and 1879, without its being noted or remembered.

For the reasons mentioned, it seems far from certain that the flood of March, 1907, has not been exceeded by any other, even since 1849; and, in fixing maximum flood rates to serve as guides in studying projects for the flood control of the Sacramento River, it seems necessary to increase somewhat the "computed" values of the flood of March, 1907. The amount of such increase cannot be logically fixed without much more study, if at all; but it seems that at least 15% should be added tentatively, making the flood rates controlling such projects at least as great as shown in Table 31.

TABLE 31.—ASSUMED MAXIMUM RATES OF FLOOD FLOW, SACRAMENTO RIVER.

Below mouth of:	Assumed by 1904 Engineering Commission. Cubic feet per second.	115% of rates computed from flood of March, 1907. Cubic feet per second.
Stony Creek	180 000 = 100%	300 000 = 167%
Feather River	190 000 = 100%	536 000 = 282%
American River	230 000 = 100%	643 000 = 280%
Cache Slough	250 000 = 100%	736 000 = 294%

Hence it seems not unreasonable to expect that any controlling works contemplated may have to deal with maximum flood rates more than two and three-quarters times as great as those assumed by the Engineering Commission of 1904. These greater floods, combined with the steadily rising river-bed, may show the whole project of flood control as outlined in 1904 to be undesirable, and may necessitate

*About in 1892—from an Exec. Doc. of 51st Congress.

Mr. Duryea. radically different methods of control. Hence, the great value of the new flood information is evident.

As interesting comparisons with the flood measurements in the paper, the writer gives measurements of the same flood as taken under his direction at points higher up on the Cosumnes and the American Rivers. Table 32 shows, in cubic feet per second per square mile, the stream flow as measured under Mr. Clapp's direction at Michigan Bar and as measured at Buck's Bar, 27½ miles farther up stream. Table 32 shows that the flood rates per square mile were much greater at the upper than at the lower station, averaging more than twice as great for the five days, March 18th to 22d, with daily means from 138% to 414% of the lower station. The greater intensity of the stream flow at the upper station was due to the heavier average rainfall above it, and to the upper area being high enough to have considerable snow, but yet not high enough to prevent it from melting easily. Reference to Table 32 shows more fully the comparative flood flows.

TABLE 32.—STORM FLOW OF COSUMNES RIVER, PER SQUARE MILE OF DRAINAGE AREA, FOR HIGH SIERRA AND TOTAL AREAS.

1907.	At Michigan Bar. Elevation, Michigan Bar = 200 ft. (about). Area = 524 sq. miles.	At Buck's Bar, 27½ miles, along river, above Michigan Bar. Elevation, Buck's Bar = 1 600 ft.* Area = 157.9 sq. miles.
	Cubic feet per second.	Cubic feet per second.
March 18th	14.50 = 100%	47.06 = 325%
19th	†62.20 = "	†86.10 = 138%
20th	17.75 = "	41.82 = 236%
21st	7.44 = "	30.75 = 414%
22d	6.30 = "	23.66 = 376%
Mean	21.64 = 100%	45.88 = 212%
February	Unknown.	7.20
March	"	13.67
March 18th-21st	‡25.47 = 100%	‡51.25 = 202%
February	Unknown.	399.0
March	"	842.5
March 18th-21st	202.2 = 100%	406.2 = 202%
Maximum rate	Unknown.	§107.0, March 19th, 7-11 A.M.

* The Buck's Bar catchment area reaches an elevation of 7 800 ft.; it includes no region of perpetual snow.
† Maximum average daily rate, both for flood and year.
‡ Maximum average 4-day rate, both for flood and year.
§ Absolute maximum rate, both for flood and year.

Table 33 shows the comparative stream flows per square mile during the flood of March, 1907, at Fair Oaks, on the American River

below the junction of the North, Middle, and South Forks, and at Slippery Ford, on the South Fork and 69 miles up stream from Fair Oaks. It is seen that the flood rates per square mile were much less at the upper than at the lower stations, less than half as great during the whole 11-day flood period, with daily means varying from 16 to 87% of the flood rates at the lower station. The smaller rate at the upper station was due to its tributary area being all above 3 900 ft. elevation and reaching to above 10 000 ft., with most of its heavy snowfall remaining unmelted throughout March. The year's stream flow Mr. Duryea.

TABLE 33.—STORM FLOW OF AMERICAN RIVER, PER SQUARE MILE OF DRAINAGE AREA, FOR HIGH SIERRA AND TOTAL AREAS.

1907.	At Fair Oaks. Elevation, Fair Oaks = about 80 ft. Area = 1 910 sq. miles.	At Slippery Ford, 69 miles along River, above Fair Oaks. Elevation, Slippery Ford = 3 900 ft. Area = 195.8 sq. miles.
	Cubic feet per second.	Cubic feet per second.
March 16th	3.56 = 100%	2.58 = 73%
17th	17.28 = "	7.93 = 46%
18th	33.10 = "	28.67 = 87%
19th	*48.70 = "	*30.07 = 62%
20th	40.35 = "	19.08 = 47%
21st	34.06 = "	8.78 = 26%
22d	28.29 = "	8.20 = 29%
23d	17.50 = "	2.91 = 17%
24th	16.92 = "	2.66 = 16%
25th	14.88 = "	4.16 = 28%
26th	13.10 = "	4.20 = 32%
Mean	24.34 = 100%	10.83 = 44½%
February	7.43 = 100%	2.58 = 35%
March	12.15 = "	5.23 = 43%
March 18th-21st	†39.05 = "	†21.63 = 55%
February	413.0 = 100%	143.7 = 35%
March	731.0 = "	321.5 = 43%
March 18th-21st	311.0 = "	171.5 = 55%
Maximum rate	Unknown.	‡32.6 March 19th. 4.30 P. M.

* Maximum average daily rate, both for flood and year.
† Maximum average 4-day rate, both for flood and year.
‡ Absolute maximum rate, both for flood and year.

measurements at Slippery Ford show that, although the highest rate of run-off occurred in March, the two months of greatest stream flow were not until May and June, when the higher snows melted.

The paper, and the writer's discussion, so far, deal with the flood of March, 1907, only as it may affect projects for the flood control of the Sacramento River. The flood-rate data of the paper have, however,

Mr. Duryea. a much more general practical application, in their bearing on the proper spillway capacities to be provided for dams in such Sierra areas.

From Table 22 and other data in the paper, the greatest rates of stream flow from the flood of March, 1907, were as shown in Table 34.

TABLE 34.—RATE OF GREATEST FLOW, IN CUBIC FEET PER SECOND, PER SQUARE MILE.

Stream.	Catchment area above gauging station, in square miles.	Flow, in Cubic Feet per Second, per Square Mile.		Percentage of maximum to average.
		Average for maximum day.	Maximum rate reached on same day.	
McCloud River	608	50.0	....	......
Upper Sacramento River	9 300	20.7	22.0	106
Upper Sacramento River	9 300	*18.9	†24.1	127½
Feather River	3 640	35.6	50.8	143
Yuba River	1 220	82.0	‡(85.3)	104
Bear River	263	106.5	......	......
American River	1 910	48.7	‡(54.0)	111
Cosumnes River	524	62.2	....	......
Calaveras River	395	66.2	....	......
Stanislaus River	935	58.1	....	......

* Deduced by writer, as hereafter explained.
† Occurred February 16th, 1907; see note to Table 7.
‡ Approximated by writer from maximum gauge heights stated.

From Tables 32 and 33, the relations between the maximum flood rate and the average rate of the maximum day's flow, as measured at two stations higher up on the Cosumnes and the American Rivers, were as shown in Table 35.

TABLE 35.

Stream.	Catchment area above gauging station, in square miles.	Flow, in Cubic Feet per Second, per Square Mile.		Percentage of maximum to average.
		Average for maximum day.	Maximum rate reached.	
Cosumnes River	157.9	86.10	*107.0	124
American "	195.8	30.07	32.6	108½

* This rate continued for 4 hours.

Table 34 is very incomplete as to the maximum rate reached, and hence it may reasonably be feared that for the rivers unobserved both the maximum rate reached and the percentage of maximum to average may have exceeded the values for the observed rivers. Tables 34 and 35 show that during the floods of March and February, 1907, controlling flood rates occurred from these Sierra areas equaling or exceeding the figures given in Table 36.

TABLE 36.—MAXIMUM RATES KNOWN TO HAVE OCCURRED DURING SIERRA FLOODS OF MARCH, 1907. Mr. Duryea.

Area furnishing flood, in square miles.	Maximum flood rates observed, in cubic feet per second, per square mile.
263	More than 107
1 220	About 85
1 910	About 54
3 640	Almost 51
9 300	*24

* Occurred February 16th, only 22 cu. ft. per sec. in March.

From Table 34 it is seen that even the large area of 3 640 sq. miles furnished a maximum observed flood rate which was 143% of the average rate for the day it occurred. Table 35 shows the corresponding relation from an area of 158 sq. miles to have been 124%; and in February, 1903, a flood on the Coyote River, in the Mt. Hamilton portion of the Coast Range Mountains, from 194 sq. miles, gave a maximum rate of 18 000 cu. ft. per sec., with an average rate for the same day of 12 000 cu. ft. per sec., or a maximum rate which was 150% of the daily rate. It is also seen that the 24 cu. ft. per sec. per sq. mile from the 9 300 sq. miles is 127½% of the average rate (18.9 cu. ft. per sec.) for the same day's flow. The 18.9 cu. ft. per sec. is not stated by the authors, however, but is derived by the writer from Table 7 and its accompanying notes, as follows:

If the gauge heights in Table 7, from 21.4 to 28.7, inclusive, are plotted with their corresponding discharges, a straight-line relation results which shows that the rate for a gauge height of 28.0 (the mean stage on February 16th, when the absolute maximum rate of 24.1 cu. ft. per sec. per sq. mile occurred) was 184 000 cu. ft. per sec. from the 9 300 sq. miles, or 18.9 cu. ft. per sec. per sq. mile. If the carrying capacity of the river in February was not materially different from that in March (and the stated rate of 224 000 cu. ft. per sec., at a gauge height of 31.0 on February 16th agrees reasonably well with the straight-line diagram for March, and indicates this to be the fact), then on February 16th, 1907, the maximum rate of discharge from this 9 300 sq. miles was 127½% (24.1 + 18.9) of the average discharge for that day.

It is seen that, even with the very limited records, maximum flood rates have been actually observed which were 143% of the mean rate for the day from an area of 3 640 sq. miles and 127½% from an area of 9 300 sq. miles. Since it is probable that with lengthening records these percentage relations will be increased, even for such large areas, and quite certain that they will be increased for smaller areas, it seems not at all improbable that maximum rates of flood flow will

Mr. Duryea. occur from any of these mixed Sierra areas of at least 150% of the greatest mean daily rate. Therefore, to the writer, it seems imprudent to expect flood rates from these areas less than those shown in Table 37, even should the greatest mean daily rates of the floods of March and February, 1907, never be exceeded, which is far from certain.

TABLE 37.—MAXIMUM SIERRA FLOOD RATES: THE LEAST WHICH MUST BE EXPECTED.

Area furnishing flood, in square miles.	Cubic Feet per Second per Square Mile.		
	Maximum rate observed.	Average rate for maximum day's flow.	Least maximum flood rate to be expected: 150% of average rate for the maximum day.
263	......	106.5	160
1 220	85.3	82.0	123
1 910	54.0	48.7	73
3 640	50.8	35.6	53
9 300	24.1	20.7	31

Passing from probable maximum flood rates to the safe capacities for which spillways should be proportioned: In the writer's opinion, spillways (even when quite full flood records are available) should always be proportioned so as to be able to pass safely at least 150% of the maximum flood rate regarded as probable, without injury to the dam. This would result in the spillway capacities shown in Table 38 as the smallest safe values for such Sierra areas.

TABLE 38.—SAFE SPILLWAY CAPACITIES: THE LEAST WHICH SHOULD BE PROVIDED FOR SUCH MIXED HIGH AND LOW SIERRA AREAS.

Area furnishing flood, in square miles.	Cubic Feet per Second per Square Mile.	
	Least maximum flood rate to be expected.	Least safe spillway capacity to be provided: 150% of least maximum flood rate.
263	160	240
1 220	123	185
1 910	73	110
3 640	53	80
9 300	31	46

The spillway capacities suggested in Table 38 are given, not as conclusive values, but merely as reasonable deductions from the authors'

Mr. Duryea.

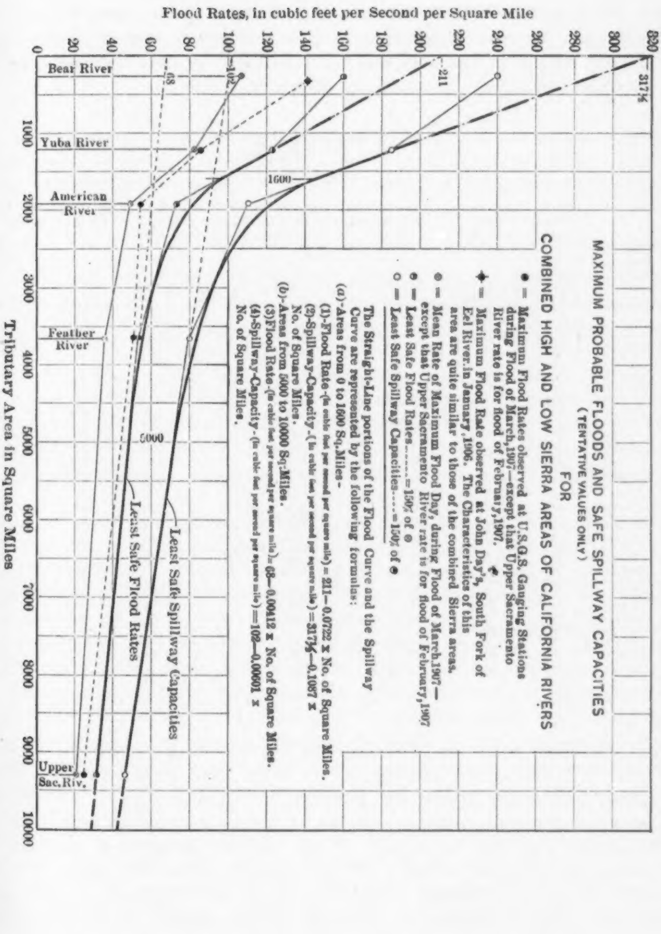


FIG. 4.

Mr. Duryea. maximum flood rates, to be tentatively used for mixed Sierra and similar areas until fuller and more conclusive data of maximum flood rates are available. The diagram, Fig. 4, puts the suggested spillway capacities in better shape for practical use, and also shows graphically their relations to the various flood-rate values of Tables 37 and 38.

In Fig. 4 the least maximum flood rate to be expected and the least safe spillway capacity to be provided are shown by curves; and the variations of these curves with changes in area are best studied by their percentage relations, as shown in Table 39.

TABLE 39.

Area furnishing flood, in square miles.	Cubic Feet per Second per Square Mile.		Percentage Relations of Flood Rates with Changing Areas; Based on the Flood Rates of the Following Areas as 100 Per Cent.						
	Least maximum flood rate to be expected.	Least safe spillway capacity to be provided.	Areas, in square miles.						
			10	200	500	1 000	2 000	5 000	10 000
			%	%	%	%	%	%	%
0	211	317½							
1	211	317	100	107	121	152	269	440	754
10	211	316	100	107	120	151	268	438	752
50	208	312	99	105	119	149	264	433	743
100	204	307	97	104	117	147	260	426	730
200	197	296	94	100	113	142	251	411	705
300	190	285	90	96	108	136	242	395	679
400	182	274	87	93	104	131	232	380	652
500	175	263	83	89	100	126	223	365	626
750	157	236	75	80	90	113	200	327	562
1 000	139	209	66	71	80	100	177	290	498
1 500	103	155	49	52	59	74	131	215	369
2 000	79	118	37	40	45	56	100	164	281
3 000	60	91	29	31	35	44	77	126	216
4 000	53	79	25	27	30	38	67	110	188
5 000	47	72	23	24	27	34	61	100	171
7 500	37	57	18	19	22	27	48	79	135
10 000	27	42	13	14	16	20	36	58	100

The flood curve and the spillway curve of Fig. 4 seem to be most open to criticism as showing, in the rates from large and from small areas, differences which are perhaps too small. This is due to the assumed constant relation (150%) of maximum rate to mean rate of the same day, for all areas. If the relation were made variable, it seems that, in view of the observed values of 143% for 3 640 sq. miles and 127½% for 9 300 sq. miles, and on account of the fact that there are few data, it should be increased for the smaller areas rather than decreased for the larger ones. That the maximum rate of the flood curve of Fig. 4 (211 cu. ft. per sec. per sq. mile, for areas of less than

10 sq. miles) under some conditions may be much exceeded, is shown by some observed flood rates on German streams, as follows:* Mr. Duryea.

Area, in square miles.	Flood rate, in cubic feet per second per square mile.	Area, in square miles.	Flood rate, in cubic feet per second per square mile.
1.3	1 116	6.7	940
3.6	1 015	20.1	366
3.8	896	116.	265
6.1	828		

The characteristics of these floods or streams are not stated. Their flow must be much more "flashy" than that of Sierra streams. From the formula given on Fig. 4, the flood rate corresponding to an area of 116 sq. miles is

$(211 - 116 \times 0.0722) = (211 - 8) = 203$ cu. ft. per sec. per sq. mile.

Personal experience in spillway studies and flood-flow measurements in the Mt. Hamilton section of the Coast Range convinces the writer that it is there inadvisable to proportion spillways for less than 200 cu. ft. per sec. per sq. mile, even for areas as great as 200 sq. miles.† In a dam designed by him on the south fork of Eel River, 150 miles farther north in the Coast Range, the spillway was proportioned for 100 000 cu. ft. per sec. from 324 sq. miles of tributary area (309 cu. ft. per sec. per sq. mile), with an extra depth of 3 ft. added to the spillway notch merely for safety and with 5 ft. more added for possible waves.

The area tributary to the Eel River Dam has a high rainfall, and sometimes considerable snow, melting quickly. Of the total 324 sq. miles, the area of 267 sq. miles beginning about 14 miles up stream from the dam is nearly circular in shape, with the rim formed by steep mountains rising to 7 000 ft. elevation. Only very meager information regarding the precipitation and stream flow existed in 1905 when the dam was designed. The dam is a combined masonry-spillway and earth-embankment structure, and the embankment would not only be ruined if overtopped by a flood, but its washing away would lead to the return of the river to an ancient filled-up channel, thus making a reconstruction of the dam very costly. Due to the local conditions, the cost added by providing an excess of spillway capacity was comparatively small.

After as full a consideration of the flood characteristics of the area as the scanty data would permit,‡ a flood rate of 300 cu. ft. per sec. per sq. mile (about 100 000 cu. ft. per sec. from the whole area) was fixed upon as not improbable, with a possibility that this rate

* Frizell's "Water-Power," 1906, p. 43.

† *Journal*, West. Soc. of Engrs., April, 1906, p. 181.

‡ *The Engineering Record*, March 14th, 1908, p. 289.

Mr. Duryea. might be much exceeded. The spillway notch called for by the final plans is 355 ft. in length and 28 ft. in depth, but the spillway studies were based on a length of 350 ft. only. The 100 000 cu. ft. per sec. would require a depth of 19.63 ft. on a 350-ft. spillway, and a depth of 23 ft. would correspond to a flow of 127 000 cu. ft. per sec. (392 cu. ft. per sec. per sq. mile); while a depth of 26 ft. would correspond to 153 000 cu. ft. per sec. (472 cu. ft. per sec. per sq. mile), with a safety allowance of only 2 ft. in depth of spillway notch for waves. In January, 1906, a flood of 46 000 cu. ft. per sec. (141 cu. ft. per sec. per sq. mile) passed over the partially constructed dam without injuring it. From a somewhat approximate estimate, the maximum rate during the flood of March, 1907 (the highest flood of that year), was 37 000 cu. ft. per sec. (114 cu. ft. per sec. per sq. mile), though the average rate for March, 1907, was 116% of that for January, 1906.

It should be borne in mind that the great spillway capacity provided in the Eel River Dam was because of the very serious results which would follow overtopping the earth portion by a flood, and the almost total lack of local flood data when the design was made; and, as in this instance the cost of providing for safely carrying the possible floods was not greatly in excess of that of providing for the probable ones, the spillway was proportioned for the former. A spillway for an earth or rock-filled dam seems to be one of the engineering constructions in which it is often advisable to provide against possibilities rather than probabilities.

The writer disclaims any general application of Fig. 4 and Table 39. The flood rates and spillway capacities there suggested are applicable only to the mixed high and low Sierra areas from which they are derived, or to areas elsewhere having quite similar characteristics. From personal observation of many reservoirs in the high Sierras, from 5 000 to 9 000 ft. elevation, where the floods are from melting snow alone, the writer believes that, under such conditions, much smaller spillway capacities than those suggested in Fig. 4 and Table 39 are safe.

Also, no general rule for spillway capacities can be safely applied directly, even in the areas from which Fig. 4 is derived. Each individual area, there or elsewhere, should receive special study, with due consideration of its precipitation, stream flow, climate, shape, slopes, vegetation, soil, etc., and their modifying influences on such general flood rates as are shown on Fig. 4. In its ultimate conclusion, the decision of proper spillway capacity should always depend on personal engineering judgment, applied separately to each special case, and cannot safely depend on the application of any general rule. The fuller the general data available, however, the better the basis on which the judgment must be exercised; and the fuller the data and the better the methods of applying them, the safer, within practicable

limits, will be the decision in each special case—which considerations form the writer's excuse for that part of his discussion relating to spillway capacities. Mr. Duryea.

Messrs. Clapp, Murphy and Martin.

W. B. Clapp, M. Am. Soc. C. E., E. C. Murphy, M. Am. Soc. C. E., and W. F. Martin, Jun. Am. Soc. C. E. (by letter).—The purpose of the writers in presenting the data contained in their paper was to co-operate as public officials in every possible way in the solution of the intricate reclamation problems in the Sacramento and San Joaquin Valleys. The data were collected in an official capacity and upon analysis were deemed to be of sufficient importance to warrant their early presentation to the Engineering Profession, so that they might become immediately available for use by engineers and others interested in such problems on the Pacific Coast, particularly in Central California. The writers did not pose as a special board of consulting engineers, appointed to investigate all the details of the great problem and recommend the most feasible scheme of improvement. They simply presented the principal facts in their possession and made such analysis of them as appeared necessary to show their bearing on the problem at hand.

It is very gratifying to know that the paper has aroused considerable interest, and the writers wish to thank those who have prepared discussions on it. In some of these discussions, it is quite evident that the point of view of the writers was entirely overlooked. In some others, their statements are not correctly quoted and interpreted. As a whole, however, all disagreements occur in instances where only a question of individual judgment is involved, and not a question of principle.

Specifically, the discussions emphasize two points brought out in the paper. First, the volume of flood water entering the valley through the different streams, and, second, the effect of mountain storage on flood control.

In regard to the estimates of flood flow, the writers repeat: "It is believed that the estimates are quite conservative, and rather inclined to be too low than too high." They do not believe that the high-water discharges in any of the streams have been over-estimated. It is true that for the higher flood stages the discharge-curve must be extended beyond the measurements, but, in almost every case, it is produced as a tangent, when, as a matter of fact, the discharge-curve is parabolic in form and concave to the axis of discharge. Extension as a straight line, therefore, or even as a very flat curve, is on the side of conservatism, especially when the curve is well determined up to moderately high stages, as was the case in this instance.

With reference to the Feather, Yuba, and American Rivers in particular, it is quite certain that there is no over-estimate, in fact, there is excellent reason to believe that the Feather was under-

Messrs. Clapp, Murphy and Martin.

estimated, and probably the Yuba also. At Oroville on the Feather River, the gauge was destroyed some time before the crest came, so that the stage was estimated from readings on the Weather Bureau gauge below, and it is now believed that the estimates of stage, and consequently of discharge, were too low. This conclusion is borne out by the records of the Great Western Power Company's station on the North Fork of Feather River at Big Bend, above Oroville.

Mr. Grunsky finds serious fault with the estimate of run-off from the unmetered mountains and foot-hills and from the valley below the foot-hills. He seems to have forgotten, absolutely, the fact, so fully set forth in the paper, that the storm from March 17th to 20th was no "ordinary rain storm." He forgot that the precipitation in March was nearly three times the normal for the month, and that about one-third of it occurred on the three days, March 17th to 19th, accompanied by comparatively high temperature and preceded by heavy rain on the 16th; also that heavy storms occurred and serious flood conditions existed in February, and were followed by almost continuous rain from March 2d to 11th, accompanied by high water in all the streams. He also states that the flow from the Sacramento Valley was assumed at 50% of the rainfall from March 17th to 20th, when it was really assumed at 40 per cent. He does not believe it possible that a run-off of 25 cu. ft. per sec. per sq. mile could occur from the unmetered mountains and foot-hills (chiefly the former with maximum elevation of 10 000 ft., and some of the heaviest precipitation stations reported) of the Sacramento Basin, notwithstanding the fact that the mean 4-day rate was considerably greater for nearly all the streams with a large percentage of high area. It is not believed that these estimates are materially in error, and, even if they were, the total inflow into the valley would be modified by a very small percentage.

The simultaneous occurrence of a very large flood on both the Sacramento and San Joaquin Rivers, is possible, but its probability is very small, on account of the great length of these two basins. Storms of great intensity seldom extend over very large areas. The maximum rate of flow of nearly all the tributaries of the San Joaquin River was greater in 1862 than in March, 1907, but this latter is the greatest recorded flood on the lower San Joaquin that has occurred simultaneously with a very heavy flood discharge of the Sacramento River.

In regard to mountain storage, it is assumed in some of the discussions that the writers propose this as the only sure and absolute panacea for all the ills of the Sacramento Valley. Such is not the case. They have simply presented the data which were in their possession, with such analysis as suggested itself in regard to the storage possibilities for flood control. They say:

"It may be that the task of rectification and enlargement of channel necessary to pass such floods as that of March, 1907, is so great as to make it economically impossible. In such event, some auxiliary system of flood control would have to be devised. Probably no more effective and easily executed auxiliary system could be found than that of large, regulating storage reservoirs in the mountains." Messrs. Clapp, Murphy and Martin.

These statements seem to have been ignored in some of the discussions.

As controllers of floods, these reservoirs do not present a very satisfactory showing, on account of their location. Less than one-fifth of their total capacity would have been available for flood control on the four days of greatest flow, and only 8% of the combined capacity of the two largest reservoirs (Big Valley in Pit River Basin and Big Meadow in Feather River Basin) would have been available on these four days. Nevertheless, the volume that could have been stored would have had a marked effect in reducing the peak of this flood. The statement of one discussor, that the storage outlined is impracticable, and, if accomplished, will prove inefficient, insufficient, precarious, and temporary, appears to be quite dogmatic, in the light of the surveys and estimates of cost made by the Engineers of the United States Reclamation Service. The reservoirs listed in this paper have been declared thoroughly practicable from an engineering point, and also financially feasible. As to the inefficiency, insufficiency, precariousness, and temporariness of these reservoirs, it is fairly certain that time alone will have to decide, since practically all of them, and probably others, will be constructed within the not distant future. Already the construction of one in the Stony Creek Basin has been started by the Reclamation Service, and others will undoubtedly follow in due time.

Before closing this discussion, it will be instructive to indicate from the available records the probabilities of filling these reservoirs after the peak of floods has passed. A seven-year record on Stony Creek shows that the reservoirs in this basin will store the entire normal flow for February or March, but cannot ordinarily be filled after March. A five-year record on Cache Creek shows that the reservoirs in this basin will store almost the entire normal flow for January and February combined, or for March alone, and can be filled ordinarily in April and May. From a four-year record on Puta Creek, it is found that the reservoirs in its basin will store almost the total normal flow for January and February, or for March, but that they cannot be filled after March. A six-year record on Feather River indicates that the reservoirs in its basin will reduce the normal flow about half in March or April, and that they can be filled after April and May. The reservoirs on the Sacramento and Pit Rivers will reduce the normal flow in March about one-third. With all the reservoirs in operation,

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the indications are that the normal flow into the Sacramento Valley would be reduced about 44% in March.

The writers believe that the permanent and ultimate solution of the flood problem in the Sacramento Valley will be embraced in one comprehensive scheme, and executed by units as a whole, and with due regard for navigation, industrial, and irrigation interests. They repeat:

"It would seem that the ultimate solution of the flood problem in the lower portions of the Sacramento Valley is closely interwoven with the reclamation of the higher portions by irrigation. Reservoirs which would impound flood waters and reduce the peak of floods * * * would serve later as storage reservoirs from which to draw for irrigation purposes."

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

Paper No. 1085.

SUBSTRUCTURE OF PISCATAQUIS BRIDGE, AND ANALYSIS OF CONCRETE WORK.*

By G. A. Hersey, Jun. Am. Soc. C. E.

With Discussion by Messrs. George B. Francis, James H. Brace, A. L. Bowman, James Owen, James C. Boyd, and G. A. Hersey.

This paper gives a general description of the construction of the Piscataquis River Bridge, built for the Bangor and Aroostook Railroad, and also the results attained with the different classes of concrete used.

The Piscataquis Bridge was built during 1907, and is a part of the "Medford Cut-off," an extension of the Northern Maine Seaport Railroad, a branch of the Bangor and Aroostook Railroad. This extension begins at the present terminus of the Northern Maine Seaport Railroad, and runs northward about 28 miles until it again strikes the main line of the Bangor and Aroostook Railroad. It shortens the distance between the two points on the main line 4.3 miles, reduces the curvature considerably, and gives much easier grades.

The "Cut-off" crosses the Piscataquis River in the Town of Medford, and on the line of a very high horse-back—a formation peculiar to that section of the country—which was of considerable value in the construction of the railroad. The line follows the horse-back in a general direction for about 14 miles, and for 6 miles skirts along its side; it can even be said that the entire road was made from it, for, as the

* Presented at the meeting of May 6th, 1908.

northerly half of the line passes through low land, material from the horse-back was used for filling, as well as ballasting. The material in it varies from sand to coarse gravel, and, in a few instances, clay. At the river the best kind of gravel was found, and the hills on either side afforded excellent sand and stone for concrete.

The grade of the railroad is 55.5 ft. above the average water level, with about 8 ft. of water in the river.

A bridge of the deck type was adopted, with four river piers and two shore abutments of reinforced concrete. The line crosses at a bend in the river, the piers being placed at an angle of 55 degrees. The total length of the bridge is 607 ft. 10 in. About 13 tons of steel were used for reinforcement, mostly in the two abutments, there being but little placed in the tops of each of the piers.

The work was handled with a Lidgerwood cableway, 800 ft. long, placed on the center line of the bridge. This cableway was used in making all the excavation, in conveying and placing concrete, moving machinery, and, later, in erecting the temporary trestle bridge. The cableway clearly demonstrated its suitability in this case, and, for rapid and profitable work, it would be hard to find anything better. In landing the north abutment, it was necessary to go about 50 ft. into the side of a 40-ft. bank and remove about 3 000 cu. yd. With the cableway, all this material was saved and used directly for concrete and for banking coffer-dams, whereas, by almost any other method, it would have been necessary to rehandle it several times.

The concrete was all machine-mixed, and dumped into buckets which were run out under the cableway and carried to any part of the bridge. The greatest number of buckets used in one day was 182, for 9 hours' work. Each bucket held 1 cu. yd.

Crib coffer-dams, of 8 by 8-in. timber, in 8-ft. sections, were made on the river bank. Alternate sections were floored about four tiers from the bottom. These cribs were then set in place, and the floored sections were loaded with rock. The outside was covered with 2-in. planks driven into the river bottom as far as possible by hand-mauls. The cribs were then banked with earth to above the water level. These coffer-dams gave excellent satisfaction, and only in one instance was there any trouble from leakage, and that was quickly remedied by a generous use of straw and gravel. The pumping was done by five centrifugal pumps having a combined discharge of 24 in., and they were able at all times to take care of the water.

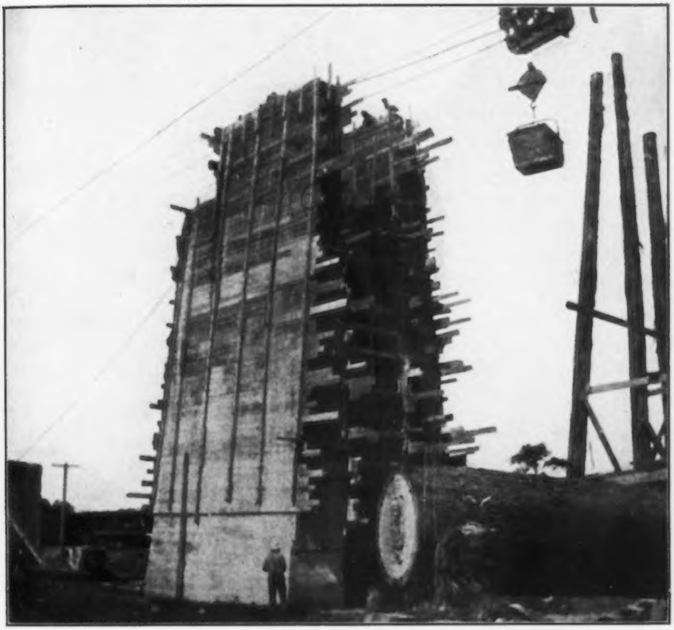

FIG. 1.—SOUTH ABUTMENT, SHOWING METHOD OF PLACING CONCRETE IN FINISHING ABUTMENT. TOTAL HEIGHT, 57 FT. (ABOVE GROUND, 45 FT.)

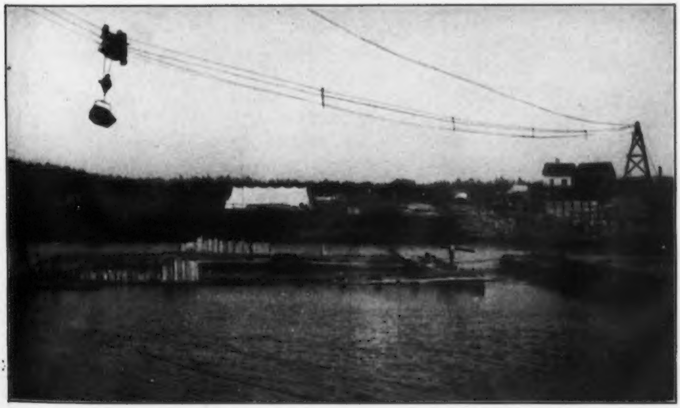

FIG. 2.—BUILDING COFFER-DAM OF PIER 3, SHOWING CABLEWAY.

The river bottom is very rocky and gravelly, and almost as hard and compact as if cemented. This feature could not have been improved upon for the foundation. Excavations were carried down to an average depth of about 5 ft. below this bottom, and all footings rested on hard gravel through which test-bars could not be driven more than 2 ft. The piers were liberally rip-rapped with the rock which had been used to load the coffer-dams, and other larger rock.

Two mixtures of concrete were used, namely: 1 : 2 : 4 for under water, and 1 : 3 : 5 for above water. Suitable gravel was found mixed with sand in about the right proportions and was used without screening. Daily tests of the aggregates were made, by volumes, so that it was known that the proper ratios were being maintained. The sand in the gravel was very clean and sharp, and free from loam and clay. The treatment of the aggregates was as follows: The specification called for measurement by volume, 1 bbl. of cement, 3 bbl. of sand, and 5 bbl. of stone, etc. Test boxes, holding half a batch of gravel, were filled with the aggregates, as placed upon the mixing platform, the contents were screened, and the sand and rock measured separately. If not in the right proportions more sand or more rock was added, as the case required. By making daily tests, and inspecting closely the materials as used, the proper proportions were maintained.

Table 1 shows the results obtained, and it is interesting to compare them with the results from mixtures made under ideal conditions, or those obtained where the ingredients were screened and graded more carefully, and the ratios were determined by weight as well as by volume.

In making the calculations in Table 1 for the number of cubic feet of concrete per barrel of cement, the quantities used in the putty coats were not considered, as so few barrels were used that they would not have had much effect on the results. The 1: 2: 4 mixture took 1.36 bbl., and the 1 : 3 : 5 mixture, 1.21 bbl. of cement per cubic yard of concrete. A perfect mixture of the 1 : 2 : 4 class would require 1.46 bbl. per cu. yd., and of the 1 : 3 : 5 class, 1.11 bbl. per cu. yd., which, as compared with the results obtained, shows that the 1 : 2 : 4 mixture fell short $\frac{1}{10}$ bbl. per cu. yd., and the 1 : 3 : 5 mixture over-ran $\frac{1}{10}$ bbl. per cu. yd. This would indicate that the cement used in the entire bridge was 83.88 bbl. more than called for theoretically.

The fact that the 1 : 2 : 4 mixture is short in cement and that the

TABLE 1.—Results Obtained in the Concrete Work for the Piscataquis River Bridge.

	1 : 2 : 4 Concrete.			1 : 3 : 5 Concrete.			Barrels of 1 : 2	Total Cubic Yards.	Total Barrels.
	Cubic Yards.	Barrels.	Cubic Feet per Barrel.	Cubic Yards.	Barrels.	Cubic Feet per Barrel.			
North Abutment	....	....	....	205.00	251.50	22.00	4.0	205.00	255.50
No. 1 Pier	174.93	237.00	19.50	233.99	280.00	22.00	3.0	411.92	520.00
No. 2 Pier	268.88	337.00	21.30	292.63	348.00	22.70	3.0	561.51	688.00
No. 3 Pier	219.83	311.00	19.00	291.91	366.00	21.60	3.0	511.74	680.00
No. 4 Pier	124.57	182.00	18.60	239.24	302.00	21.40	1.5	363.81	485.50
South Abutment	70.00	101.00	18.70	429.03	508.50	22.80	3.0	499.03	612.50
Totals	858.21	1 168.00	19.80	1 694.80	2 056.00	22.20	17.5	2 553.01	3 241.50

FIG. 1.—GENERAL VIEW, LOOKING SOUTH. NORTH ABUTMENT AND PIERS 1 AND 4 FINISHED. PIER 2 NEARLY COMPLETE, AND PIER 3 BEING EXCAVATED.

FIG. 2.—GENERAL VIEW OF COMPLETED BRIDGE.

1 : 3 : 5 mixture has a surplus, may be accounted for in two different ways: First, the aggregates were not graded properly, so that the voids were not completely filled; second, the quantities of the aggregates used for the different mixtures were not always exactly the same. The material was loaded into the mixer by wheel-barrows, the necessary quantity for a wheel-barrow for each mixture was determined, and they were all supposed to be loaded the same; in this, however, there was bound to be some variation.

For 1 bbl. of cement, or 4 bags (all the cement being in bags) 8 wheel-barrows of the aggregate were used in the 1 : 3 : 5 mixture, and 6 wheel-barrows of the aggregate in the 1 : 2 : 4 mixture. A check was had on the quantities used by the space occupied by the completed batch in the bucket, after being dumped from the mixer, as the buckets each held 1 cu. yd., it was found by test samples just how full they should be for the two different mixtures. Then, again, the aggregates not being uniformly graded by Nature would have a tendency to throw the resultant cubic feet per barrel, under or above the theoretical results obtained from perfect mixtures. It seems, however, that where suitable ingredients can be found in their natural state, free from loam and clay—although a small percentage of either will not decrease the strength of the concrete—that as good work at less cost per yard can be obtained as where the sand is screened from the gravel and they are again mixed artificially; for, when the ingredients go into the mixer in their natural state, the machine has to do less work in mixing the cement with them, than when the sand, rock, and cement go in separately, for the mixer has not only to mix the cement in, but the sand and rock as well. Thus, with a mixer running for the same time, under the two different conditions, it would seem that a better mixture could be made from the natural ingredients. There were instances during the progress of the work when the resultant mixtures agreed exactly with the required quantity of cement per barrel. This would indicate that in such cases the voids were completely filled, the sand filling the interstices of the rock and the cement those of the sand.

The piers were designed with round noses, having a slight batter. At a point well above high water, on the front ends, a circular breakwater was made, having a batter of 2 : 3 and extending down to within 3 ft. of the footing course. The forms for these noses were made

of 2-in. plank, about 4 in. wide, sections being built up in 8-ft. lengths as the concrete advanced. With plank of this width, the circular form could be made very readily, and with very smooth surfaces.

The concrete was mixed wet, no tamping being required, other than the shoveling over it received after being dumped from the bucket. Care was taken, however, that the sides of the forms were well worked around with shovels, which kept the rock back and allowed the soft material to come to the outside. The piers when stripped and dry received a coat of whitewash. The writer is not wholly convinced of the worth of this coat for work of this class, as it usually cracks and peels off. Where a good surface has been obtained, he would prefer to omit the whitewash coat.

Although the season was unusually wet, the progress of the work was delayed only for a few days. The river is affected rapidly by the rains, there being no storage in its water-shed, and the water rises and falls quickly. High water was encountered only once, when the second and third piers were first started, but the only damage was the washing away of a little of the coffer-dam embankment.

In the construction of the temporary falsework, piles were driven from a driver on a barrel raft. In no case could the piles be driven more than 5 ft., and the average was about 3 ft. This proved fully the firmness of the entire river bottom, as found during the excavation at the pier locations. Most of the piles were furnished with steel points. For absolute safety, piles should not be driven without some protection for the point, no matter through what kind of ground, as one can never tell what material a pile is to pass through.

The contractor for the concrete work was Mr. J. B. Mullen, who has had wide experience in similar work. Moses Burpee, M. Am. Soc. C. E., is Chief Engineer of the Bangor and Aroostook Railroad. W. S. McFetridge, M. Am. Soc. C. E., was Engineer of Construction, in charge of the "Medford Extension," and the writer was Engineer in Charge at the bridge.

DISCUSSION.

George B. Francis, M. Am. Soc. C. E.—About six years ago, some concrete engine and generator foundations were built under the speaker's direction, one of which it was necessary to remove after it had been standing two or three months, and in the removal there was opportunity to observe the effect of the stoppage of the placing of the concrete over night—the original construction occupying several days. Mr. Francis.

This particular foundation was approximately 30 ft. long, 20 ft. wide and 15 ft. high, and the cause of its removal was a change of the type of engine and generator which it was to support, the new ones requiring a totally different arrangement of anchor bolts.

The concrete was supposed to be a monolithic structure, and it had become so thoroughly set that it required the use of dynamite to break it up.

In separating, it came apart on horizontal lines corresponding to each day's work, showing distinctly that there was only moderate cohesion of one day's work with that of another, although there was no evidence of these seams on the exterior surface. The vertical breaks, as might be expected, were ragged and through the homogeneous mass.

In recent work it has been the custom to place the concrete for such structures without intermission until completed.

Mr. Hersey speaks of using some sort of a metal shoe on all piles for foundation work. The speaker does not agree with him in this, as there are many cases, where piles are required, where it is quite plain that they will have to penetrate known strata, and that a metal shoe or protection is unnecessary and a great waste of money.

In many cases where piles cannot be driven without shoes, it is pretty good evidence that they are not needed. When they can be driven only 4 or 5 ft., it is better to deepen the excavation and not use them at all.

James H. Brace, M. Am. Soc. C. E.—In the speaker's practice, the question of joints has come up in the construction of arches. In one case (a very flat, thick, and heavily reinforced concrete arch), as the contractor did not wish to work at night, the arch was divided into voussoirs by radial bulkheads. The size of each voussoir was about equivalent to one-half the daily capacity of the concrete plant. Work was started at the skewbacks, and one section on each side of the arch was filled during the day. This secured a balanced loading on the centers, and radial joints. The results were satisfactory. Wherever possible, however, the speaker believes it is best to work continuously until the arch is completed. When this is done, it is well to build up the concrete as nearly as possible along radial lines, Mr. Brace.

Mr. Brace. in order to avoid bad joints in case of some unavoidable delay. This is particularly important in lining tunnels, as the work approaches the top of the arch. It always requires much more time to place the concrete in the key, and the concrete on either side is apt to take considerable set before the work is completed.

It is thought that by "whitewashing" the author means a layer of neat cement grout brushed over the finished work. The speaker has seen this used, and thinks that it generally improves the appearance of the work. It will sometimes peel off, but is more apt to adhere to the wall than a plaster coat.

Mr. Bowman. A. L. Bowman, M. Am. Soc. C. E.—In work of this kind, to secure the best results in quality and appearance, a wet concrete should be used, and to secure the advantages of a monolithic structure, within certain lines where possible, the concreting should be carried on continuously, day and night, until finished.

A few years ago the speaker had occasion to build some reinforced concrete railroad arches. The abutments and piers were carried up to the springing lines and there stopped. When the arch rings were started the work was carried on continuously, night and day, until finished.

In one of the spans—a double-track, 54-ft. skew arch—the concreting was started simultaneously from each skewback, and was built in 60 hours of continuous work. The concrete was mixed very wet, and, owing to the number and location of the reinforcing rods, very little tamping could be done. The forms were made of dressed lumber, and the inside faces were coated with "Petrolatum" as soon as erected, a second coat being applied just before the concreting was started.

To secure a smooth surface, the concrete was worked thoroughly against the forms with forks, so as to force the stone back into the body of the ring and flush the cement mortar to the surface. The forms were set ½ in. high at the crown, and, after the arch ring was completed, were left in place for about two weeks, after which they were lowered by small amounts every few days until they were clear of the arch.

When the centers were entirely removed, the inner face of the arch presented a very smoothly-finished, uniformly-colored surface, which only had to be touched up and rubbed down in a few spots.

There were no hair cracks visible, and very satisfactory results were secured without using any cement wash or whitewash coat.

Mr. Owen. James Owen, M. Am. Soc. C. E.—In the construction of piers and concrete work there is one feature which has not received much attention, although it has been a matter of some consideration, and that is what is known as the over-night bond. It is a generally accepted fact that it is very difficult to obtain a perfectly homogeneous connection

with work finished over-night and built on the next day, on account of expansion. In a large proportion of concrete construction, the joint, where one section is put upon the other, is very noticeable when the work is completed, and many methods of overcoming this difficulty have been tried. Mr. Owen.

The speaker tried the experiment of making the last run of the machine very wet, and found this somewhat efficacious. This run was so wet that the next morning the water had all collected on the top of the pier, with very slight setting on the surface of the concrete. The water was swept off and the new joint made, and thus the difficulty was overcome, in a measure; but, if there are such joints in pier construction between high and low water, such trouble is likely to occur.

A great many engineers do not allow any intermission, the work being carried on continuously night and day. This method eliminates entirely the bond question, but it is somewhat more costly.

The speaker's attention was first called to the bond question in the case of a Paterson bridge which was carried away by a freshet. On examination, the break was found to have occurred on a line of work where two sections had not been thoroughly bonded in construction.

JAMES C. BOYD, M. AM. SOC. C. E. (by letter).—Some years ago the writer was in charge of bridge construction for the Bangor and Aroostook Railroad, and therefore he has read with much interest Mr. Hersey's paper on the Piscataquis Bridge. At that time a bridge, very similar in character and construction, was built at what is known as the Second Crossing of the Fish River. The close similarity in some of the results obtained on the Fish River and this Piscataquis work is rather striking. On the Fish River Bridge, the concrete was mixed by hand, the aggregate being bank gravel taken from a near-by cut. This gravel was not screened, but was tested in a rather crude manner, under the writer's direction, by periodically taking sample buckets of the gravel and ascertaining the voids by noting the quantity of water required to fill them, a correction being made in the aggregate by the addition of either sand or gravel as the case required. In the concrete it was desired to obtain the equivalent of a 1 : 3 : 5 mixture, and the records of this work show that two bridge abutments, containing 1 668 cu. yd., required 2 034 bbl. of cement, giving an average of 1.21 bbl. of cement per cubic yard of concrete. This work was performed by day's labor by the railroad company, and the cost, exclusive of overhead charges, but including foundation caissons and pumping, forms and tools, was $7.76 per cu. yd. of concrete, cement costing $3.00 per bbl. delivered at the site. Mr. Boyd.

In a similar bridge, known as the First Crossing of the Fish River, built at the same time and in the same manner, 3 216 cu. yd. of concrete required 4 060 bbl. of cement, giving an average of 1.26 bbl. per cu. yd. of concrete.

Mr. Boyd. The author states that, for absolute safety, it is necessary to furnish all piles with point protection. The writer feels that this is not strictly true, although, in material found in the river bottoms in the northern sections of Maine, it is no doubt the safest course to pursue. However, in many places where soft material is found and the piles depend on skin friction for their supporting power, as well as in many places where the character of the foundation is well known, it would hardly apply.

Mr. Hersey. G. A. HERSEY, JUN. AM. SOC. C. E. (by letter).—It may not always be necessary, perhaps, to prepare soft wooden piles with steel points, but, from two instances which have come under the writer's observation, it would seem to be the safer course, especially under conditions such as are met with in Maine. The piles used on the Piscataquis River Bridge were for a temporary wooden trestle which was in use only a short time. The river bed was so rocky, and penetration was so difficult, that piles driven without points were soon battered and broomed, and, even if one did happen to drive solid, on being pulled up, the lower 3 to 5 ft. were found to be practically destroyed. On the other hand, where a steel point was used, the driving was much easier, and, upon removal, the pile was as good as new, the steel point preventing all brooming. The points are easily put on and removed, and suitable for service elsewhere.

In another instance, on a double-track trestle over a bog, after the piles were driven some distance, they struck a ledge and slid out of place. The use of steel points overcame the difficulty.

While satisfactory results may be obtained with a "whitewash coat," the writer believes that, in railroad culvert work, abutments, and piers, outside of towns and cities, it is not necessary; provided, however, proper care is exercised in making the forms, in placing the concrete, and in using wet concrete. A coating of oil may also be applied to the forms at projections and, possibly, to the sides.

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TRANSACTIONS

Paper No. 1086

THE SEMICIRCULAR MASONRY ARCH.

By A. E. Lindau, Assoc. M. Am. Soc. C. E.

Since the introduction of reinforced concrete construction, the building of arches has increased to such an extent that simple, approximate methods of finding bending moments and thrusts, which are sufficiently accurate for all practical purposes, will undoubtedly be appreciated by those interested in this branch of engineering.

Nothing simpler in arch analysis has been proposed than the coefficients for parabolic arches by the late C. E. Greene, M. Am. Soc. C. E. The fact that they are confined to curves of one type is, of course, a serious limitation, and the purpose of this paper is to extend this method to the analysis of the semicircular arch with fixed ends, basing the coefficients for bending moments and thrusts on the ordinates to the equilibrium polygons given by Professor Greene.

Horizontal Thrust and Reaction.—The center line of the arch, Fig. 1, is assumed to be a semicircle and divided into equal segments every 10°, beginning with zero at the crown. It is proposed to express the bending moment at any one of these points, for a load of W at any other point, in terms of R, the radius, and W, the load.

Let $N\ C\ Q$, Fig. 1, represent the equilibrium polygon for the load, W, at an angle, a, to the right of the crown, y_1 being the ordinate at the left abutment, y_0 the ordinate to the apex, and y_2 the ordinate at the right abutment. The bending moment at any point is, of course, equal to the horizontal thrust multiplied by the vertical distance from

the equilibrium polygon to the center line of the arch, or, from Fig. 1, $H \times E\,F$.

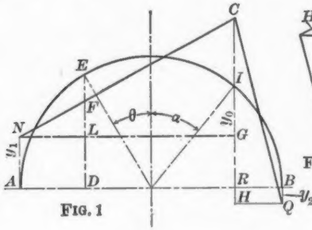

FIG. 1

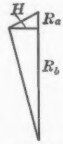

FIG. 1a

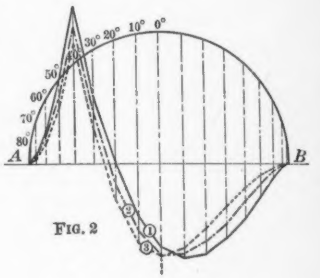

FIG. 2

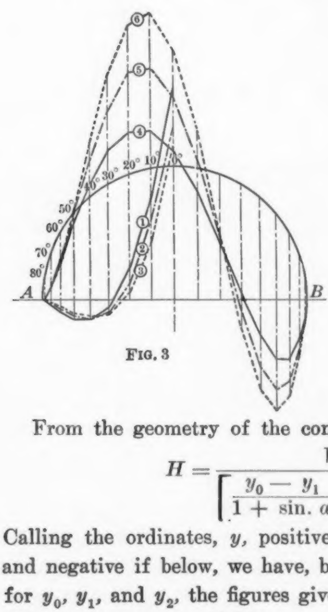

FIG. 3

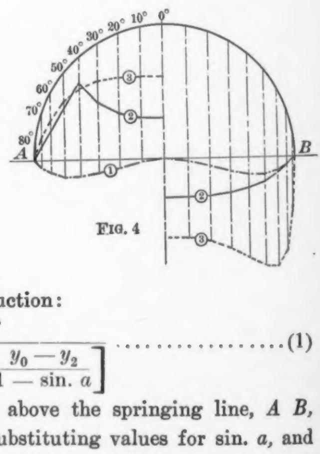

FIG. 4

From the geometry of the construction:

$$H = \frac{W\,R}{\left[\dfrac{y_0 - y_1}{1 + \sin.\,a} + \dfrac{y_0 - y_2}{1 - \sin.\,a}\right]} \qquad \text{(1)}$$

Calling the ordinates, y, positive if above the springing line, $A\ B$, and negative if below, we have, by substituting values for sin. a, and for y_0, y_1, and y_2, the figures given on page 94 of Greene's "Arches," the following values of H:

a	0°	10°	20°	30°	40°	50°	60°	70°	80°
y_0..	1.330R	1.326R	1.316R	1.298R	1.275R	1.245R	1.210R	1.170R	1.125R
y_1..	0.241 "	0.288 "	0.326 "	0.360 "	0.387 "	0.413 "	0.434 "	0.455 "	0.475 "
y_2..	+0.241 "	+0.183 "	+0.108 "	+0.011 "	−0.125 "	−0.330 "	−0.665 "	−1.333 "	−3.319 "
H..	0.459 W	0.441 W	0.386 W	0.313 W	0.224 W	0.139 W	0.069 W	0.0238 W	0.0035 W

Referring to Fig. 1*a*, we have, for the value of the reaction at the left abutment:

$$R_a = \frac{y_0 - y_1}{R\,(1 + \sin.\,a)}\,H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

$$R_b = W - R_a.$$

Again substituting values for y_0, y_1, and the above values of H, we have the following values of R_a and R_b:

Point.	0°	10°	20°	30°	40°	50°	60°	70°	80°
R_a	0.5 W	0.458 W	0.382 W	0.294 W	0.199 W	0.115 W	0.054 W	0.017 W	0.002 W
R_b	0.5 W	0.542 W	0.618 W	0.706 W	0.801 W	0.885 W	0.946 W	0.983 W	0.998 W

Bending-Moment Coefficients for Vertical Loads.—Since the value of H is given as a coefficient multiplied by W, the next step is to determine the distance, $E\,F$, in terms of R, for all points of the arch when the load, W, is applied at any point, a, to the left or right of the crown.

Referring again to Fig. 1, the distance, $L\,F$, is given by the ratio, $\frac{L\,F}{N\,L} = \frac{C\,G}{N\,G}$, from which:

$$L\,F = \frac{1 - \sin.\,\theta}{1 + \sin.\,a}\,(y_0 - y_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(3)$$

for all points to the left of the load,

$$L\,F = \frac{1 + \sin.\,\theta}{1 + \sin.\,a}\,(y_0 - y_1) \quad \ldots\ldots\ldots\ldots\ldots\ldots(3a)$$

for all points to the left of the load up to the crown,

$$L\,F = \frac{1 - \sin.\,\theta}{1 - \sin.\,a}\,(y_0 + y_2) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

to the right of the load,

$D\,E = R\cos.\,\theta$, and $D\,F = y_1 + L\,F$; consequently

$$-\,E\,F = -\,R\cos.\,\theta + y_1 + \left[\frac{1 \mp \sin.\,\theta}{1 + \sin.\,a}\,(y_0 - y_1)\right] \ldots\ \ldots(5)$$

and

$$-\,E'\,F' = -\,R\cos.\,\theta + y_2 + \left[\frac{1 - \sin.\,\theta}{1 - \sin.\,a}\,(y_0 - y_2)\right] \ldots\ldots(6)$$

Considering then, that for the load, W, in any particular position, a, to the left or the right of the crown, the term, $\sin.\,a$, is constant, we can, by giving definite values to the angle, θ, find the values of $E\,F$ in terms of R.

To illustrate the manner in which this computation may be accomplished, assume the load, W, to be placed at the angle, $a = 20°$, to the right of the crown.

Then with $a = 20°$, $H = 0.386\ W$.

$$\frac{y_0 - y_1}{1 + \sin. a} = 0.7376 \qquad \frac{y_0 - y_2}{1 - \sin. a} = 1.8359$$

TABLE 1.

Any point θ.	Cos. θ.	1 − sin. θ.	*L F.*	*E F.*	Bending moment, $H \times E F$.
Right abutment	0.000	0.000	0.000*R*	+ 0.108*R*	+ 0.041688 *RW*
80......................	0.17365	0.01519	0.0278874*R*	− 0.0377676*R*	− 0.0145763 "
70......................	0.34202	0.06031	0.1107237 "	− 0.16783 "	− 0.0450782 "
60......................	0.50000	0.13397	0.2459568 "	− 0.14604 "	− 0.0563714 "
50......................	0.64279	0.23396	0.4295195 "	− 0.10527 "	− 0.040634 "
40......................	0.76604	0.35721	0.655805 "	− 0.002235 "	− 0.00086271 "
30......................	0.86603	0.5000	0.91795 "	+ 0.15992 "	+ 0.061729 "
20......................	0.93969	0.65798	0.1208 "	+ 0.37631 "	+ 0.145255 "
10......................	0.98481	0.82635	0.866778 "	+ 0.206968 "	+ 0.0798896 "
0......................	1.0000	1.0000	0.73768 "	+ 0.06368 "	+ 0.0245805 "
10......................	0.98481	0.82635	0.60958 "	− 0.4923 "	− 0.01900278 "
20......................	0.93969	0.65798	0.485378 "	− 0.22831 "	− 0.0495276 "
30......................	0.86603	0.50000	0.36884 "	− 0.17119 "	− 0.0680793 "
40......................	0.76604	0.35721	0.263506 "	− 0.17653 "	− 0.0681408 "
50......................	0.64279	0.23396	0.1725876 "	− 0.14420 "	− 0.0556612 "
60......................	0.50000	0.13397	0.0988269 "	− 0.057517 "	− 0.0290156 "
70......................	0.34202	0.06031	0.044489 "	+ 0.028469 "	+ 0.010989 "
80......................	0.17365	0.01519	0.011205 "	+ 0.153555 "	+ 0.0592724 "
Left abutment.........	0.0000	0.0000	0.0000 "	+ 0.326 "	+ 0.125836 "

The last column of Table 1 gives the bending moments, in terms of R and W, at all points of the span, when the load, W, occupies the position 20° to the right of the crown; similarly, for any other point, a. By arranging a table for all points, a, beginning at the top with the right abutment, continuing through to the left abutment, writing values similar to the last column of Table 1 across the page for each point, we have Table 2, giving bending moments in terms of R and W, and a coefficient, C, for various points of the arch and various positions of the vertical load, W.

$$M = C \times R \times W.$$

It is to be noted that if the loads are placed at the points indicated in the first column of Table 2, bending-moment coefficients are found in the following columns. For example, a load placed 10° to the right of the crown causes a bending moment of $+ 0.0146\ R\ W$ at a point 10° to the left of the crown.

Bending-Moment Coefficients for Horizontal Loads.—By a method similar to the preceding one, coefficients for bending moments caused

TABLE 2.—BENDING-MOMENT COEFFICIENTS, C, FOR VERTICAL LOADS.

Load at: \ M. at:	90°	80°	70°	60°	50°	40°	30°	20°	10°	0°	
R. Abut.	L. Abut.										
80°	+ 0.0016	+ 0.0010	+ 0.0005	+ 0.0001	− 0.0003	− 0.0006	− 0.0008	− 0.0008	− 0.0008	− 0.0007	Times RW.
70°	+ 0.0108	+ 0.0068	+ 0.0032	+ 0.0001	− 0.0024	− 0.0043	− 0.0054	− 0.0068	− 0.0054	− 0.0042	" "
60°	+ 0.0301	+ 0.0185	+ 0.0081	− 0.0007	− 0.0077	− 0.0127	− 0.0156	− 0.0161	− 0.0144	− 0.0104	" "
50°	+ 0.0574	+ 0.0343	+ 0.0138	− 0.0033	− 0.0166	− 0.0257	− 0.0302	− 0.0301	− 0.0254	− 0.0161	" "
40°	+ 0.0867	+ 0.0496	+ 0.0174	− 0.0091	− 0.0290	− 0.0417	− 0.0468	− 0.0441	− 0.0339	− 0.0162	" "
30°	+ 0.1127	+ 0.0613	+ 0.0173	− 0.0178	− 0.0431	− 0.0578	− 0.0614	− 0.0506	− 0.0352	− 0.0063	" "
20°	+ 0.1258	+ 0.0593	+ 0.0110	− 0.0290	− 0.0557	− 0.0681	− 0.0661	− 0.0495	− 0.0190	+ 0.0246	" "
10°	+ 0.1270	+ 0.0564	− 0.0003	− 0.0412	− 0.0652	− 0.0715	− 0.0599	− 0.0308	+ 0.0146	+ 0.0760	" "
0°	+ 0.1106	+ 0.0385	− 0.0162	− 0.0519	− 0.0675	− 0.0624	− 0.0370	+ 0.0082	+ 0.0716	+ 0.1515	" "
10°	+ 0.0807	+ 0.0134	− 0.0333	− 0.0586	− 0.0601	− 0.0392	+ 0.0088	+ 0.0677	+ 0.1505	+ 0.0760	" "
20°	+ 0.0417	− 0.0146	− 0.0451	− 0.0564	− 0.0406	− 0.0009	+ 0.0617	+ 0.1453	+ 0.0799	+ 0.0246	" "
30°	+ 0.0034	− 0.0387	− 0.0550	− 0.0451	− 0.0093	+ 0.0515	+ 0.1352	+ 0.0790	+ 0.0322	− 0.0063	" "
40°	− 0.0280	− 0.0536	− 0.0517	− 0.0224	+ 0.0334	+ 0.1140	+ 0.0743	+ 0.0387	+ 0.0089	− 0.0162	" "
50°	− 0.0459	− 0.0558	− 0.0370	+ 0.0100	+ 0.0837	+ 0.0585	+ 0.0353	+ 0.0147	− 0.0026	− 0.0161	" "
60°	− 0.0462	− 0.0434	− 0.0106	+ 0.0493	+ 0.0365	+ 0.0244	+ 0.0133	+ 0.0036	− 0.0044	− 0.0104	" "
70°	− 0.0317	− 0.0209	+ 0.0197	+ 0.0153	+ 0.0110	+ 0.0070	+ 0.0034	+ 0.0002	− 0.0023	− 0.0042	" "
80°	− 0.0011	+ 0.0032	+ 0.0026	+ 0.0020	+ 0.0014	+ 0.0008	+ 0.0003	− 0.0001	− 0.0004	− 0.0007	" "
L. Abut.											

by horizontal forces can be found. Solving the equations on pages 119-123 of Greene's "Arches," the values for X_0, X_1, X_2, H_1, H_2, and P, etc., in Table 3 are obtained.

TABLE 3.

a	X_0	X_1	X_2	H_1	H_2	P
0	0.0000R	0.5708R	0.5708R	0.5000H	0.5000H	0.3183H
10	0.0042 "	0.5924 "	0.5976 "	0.4979 "	0.5027 "	0.3087 "
20	0.0337 "	0.6500 "	0.6932 "	0.4834 "	0.5166 "	0.2811 "
30	0.1126 "	0.7355 "	0.8920 "	0.4474 "	0.5526 "	0.2387 "
40	0.2635 "	0.8438 "	1.2573 "	0.3853 "	0.6147 "	0.1868 "
50	0.5058 "	0.9713 "	1.9162 "	0.2998 "	0.7602 "	0.1315 "
60	0.8556 "	1.1158 "	3.1673 "	0.2006 "	0.7994 "	0.0795 "
70	1.3248 "	1.2765 "	5.9089 "	0.1036 "	0.8964 "	0.0372 "
80	1.9189 "	1.4506 "	14.6411 "	0.0294 "	0.9706 "	0.0096 "
90	....	∞	∞	0.0000 "	1.0000 "	0.0000 "

The vertical thrust, P, constant throughout the span, is analogous to the horizontal thrust for vertical loads; and the bending moment for the horizontal force, H, is equal to P multiplied by the horizontal distance from the center line of the arch to the equilibrium polygon determined by the position of H. These horizontal distances are evaluated in much the same manner as the vertical ordinates, $E\ F$, and multiplying by the proper value of P and tabulating gives Table 4, from which the bending moment at any point due to the horizontal force, H, is $M = C \times H \times R$.

Influence Lines.—If the figures in any column, Tables 2 or 4, be plotted to scale and the points connected by lines, the resulting curve may be called the influence line for this particular point in the span. In Fig. 2 the full-line curve marked 1 shows the influence line for the point 40° to the left of the crown, plotted from Table 2, ordinates below the line, A-B, being negative and those above positive. From Fig. 2 it is evident that the maximum positive moment occurs when the load is directly over the point, and the maximum negative moment with the load near or slightly to the right of the crown, while loads near the abutments exercise very little influence.

In Fig. 3, Curves 1 and 4 are influence lines for the crown and abutment, B, respectively. Fig. 4 indicates influence lines for horizontal forces. Curve 1 is for the crown, showing that the bending moment for this point is negative for all positions of the load. Curve 2 is for the 40° point, and Curve 3 for the abutment, B. Curves 2 and 3 appear to be discontinuous at the crown, which is due to the

TABLE 4.—COEFFICIENTS FOR HORIZONTAL LOADS.

W. on	90°	80°	70°	60°	50°	40°	30°	20°	10°	0°	
80°	− 0.1405	+ 0.0279	+ 0.0225	+ 0.0171	+ 0.0120	+ 0.0072	+ 0.0029	− 0.0008	− 0.0038	− 0.0059	Times HW.
70°	− 0.2198	− 0.0649	+ 0.0842	+ 0.0651	+ 0.0466	+ 0.0326	+ 0.0136	+ 0.0001	− 0.0108	− 0.0188	" "
60°	− 0.2521	− 0.1145	+ 0.0165	+ 0.1370	+ 0.1004	+ 0.0659	+ 0.0345	+ 0.0071	− 0.0153	− 0.0322	" "
50°	− 0.2520	− 0.1324	− 0.0205	+ 0.0804	+ 0.1672	+ 0.1141	+ 0.0653	+ 0.0225	− 0.0132	− 0.0406	" "
40°	− 0.2349	− 0.1310	− 0.0359	+ 0.0475	+ 0.1166	+ 0.1698	+ 0.1041	+ 0.0462	− 0.0038	− 0.0409	" "
30°	− 0.2129	− 0.1206	− 0.0384	+ 0.0314	+ 0.0864	+ 0.1250	+ 0.1462	+ 0.0755	+ 0.0152	− 0.0331	" "
20°	− 0.1948	− 0.1095	− 0.0351	+ 0.0257	+ 0.0714	+ 0.1004	+ 0.1119	+ 0.1056	+ 0.0365	− 0.0197	" "
10°	− 0.1845	− 0.1020	− 0.0314	+ 0.0221	+ 0.0321	+ 0.0899	+ 0.0960	+ 0.0842	+ 0.0549	− 0.0063	" "
0°	....	....	....	....	....	....	....	....	....	....	" "
10°	+ 0.1829	+ 0.1012	+ 0.0312	− 0.0247	− 0.0649	− 0.0882	− 0.0939	− 0.0818	− 0.0523	− 0.0063	" "
20°	+ 0.1827	+ 0.1030	+ 0.0343	− 0.0214	− 0.0623	− 0.0872	− 0.0954	− 0.0867	− 0.0611	− 0.0197	" "
30°	+ 0.1756	+ 0.1015	+ 0.0370	− 0.0161	− 0.0561	− 0.0818	− 0.0925	− 0.0877	− 0.0677	− 0.0331	" "
40°	+ 0.1576	+ 0.0936	+ 0.0371	− 0.0100	− 0.0464	− 0.0708	− 0.0827	− 0.0816	− 0.0675	− 0.0409	" "
50°	+ 0.1277	+ 0.0777	+ 0.0331	− 0.0045	− 0.0342	− 0.0549	− 0.0661	− 0.0674	− 0.0588	− 0.0406	" "
60°	+ 0.0888	+ 0.0552	+ 0.0250	− 0.0008	− 0.0215	− 0.0364	− 0.0451	− 0.0473	− 0.0430	− 0.0322	" "
70°	+ 0.0475	+ 0.0301	+ 0.0143	− 0.0007	− 0.0103	− 0.0185	− 0.0235	− 0.0253	− 0.0237	− 0.0188	" "
80°	+ 0.0139	+ 0.0090	+ 0.0044	− 0.0005	− 0.0027	− 0.0052	− 0.0067	− 0.0074	− 0.0071	− 0.0059	" "

change in direction of the forces at this point. If the forces were assumed to be in the same direction from A to B, Curves 2 and 3 would be continuous, and the ordinates would be of the same sign throughout.

It is to be noted that Tables 2 and 4 are both subject to the limitation that the thickness of the arch ring is constant from crown to abutments. Arches, of course, are not built with constant thickness, for various reasons, therefore it would be of interest to find what influence, if any, the increase in the thickness of the arch ring toward the abutment may exert on the bending moments and thrusts.

Effect on Bending Moments for Variation in Ring Thickness.—With this object in view, two special cases were investigated by using Professor Howe's summation formulas, given in his treatise, "Symmetrical Arches." In one case the thickness at the abutment was made one and one-half times that at the crown, and in the other twice the crown thickness. With these assumptions, the case of the semicircular arch of any radius, R, can be investigated, because the moment of inertia at any section becomes a multiple of that at the crown, and the thickness, t, disappears in the formulas for H, M_1, and M_2.

Referring again to Figs. 2 and 3, this effect of change in section on the influence lines is shown by the dotted and the dot-and-dash lines, the dotted line in each case showing the effect of the abutment thickness being twice that of the crown, and the dot-and-dash lines one and one-half times as much.

It will be observed that, for the crown and also for the 40° point, the effect of increasing the ring thickness is not so marked, while, for the abutment, the bending moments for loads at certain points are widely different for the three cases.

Effect on the Horizontal Thrust for Variation in Ring Thickness.—That the thickening of the arch at the abutment also influences the horizontal thrust is evident from Fig. 5, in which three curves are shown, the ordinates to which indicate the coefficient by which W, at any point, is multiplied to give the horizontal thrust, H. Again, Curve 1 refers to the case of constant thickness, Curves 2 and 3 to the ratios, $1\frac{1}{2}$ and 2, respectively. It will be observed that while the coefficients for loads near the crown increase as the thickness at the abutment increases, the reverse is true from the 40° point to the

abutment, so that, taking the arch as a whole, that is, the total thrust of the span, the results for the three cases are not so far apart as might be supposed from the figure.

The point of importance to the designer is whether these variations will materially affect the maximum moments and thrusts; or, still more to the point, the maximum fiber stresses at any section. To throw some light on this phase of the matter will require the analysis of a particular example.

Example.—Assume a 50-ft. clear-span arch for railroad traffic, with 3 ft. of fill over the crown, the live load to be considered equivalent to 1 000 lb. per sq. ft. and the horizontal pressure at any point one-third of the vertical. In Fig. 6 are drawn to scale several bending-moment curves for this arch. Curve 1 shows the bending moments at all points in the span for the dead load of the arch ring and the earth fill, considering vertical loads only and the ring thickness constant. Curve 2 shows the same load, with the abutment thickness twice that at the crown; Curve 3 shows the live load only from *B* up to and including the 20° point to the left of the crown; Curve 4 shows the same live load, but the ring thickness is the same as in Curve 2.

Apparently, the effect of thickening the abutment is to decrease the bending moment over the greater part of the span, at the expense of the abutment where there is a great increase of moment, a result which might be expected because of the increased rigidity of the structure at this point.

Maximum Bending Moments.—Fig. 7 shows all the positions of live load giving maximum moments: Curves 2 to 7. The heavy lines at the top of the figure indicate the extent of the loading. These curves show the moments for vertical load only. The dead-load moments are given by Curves 1 and 8, Curve 1 for the vertical, and Curve 8 for the horizontal forces.

It is evident by inspection that the vertical dead-load moments are practically neutralized by the moments caused by horizontal earth pressure. This is shown more clearly in Fig. 8, where Curve 1 is the result of combining Curves 1 and 8 in Fig. 7. Curve 2, in Fig. 8, corresponding to a live load, extending from the abutment, *B,* up to and including the 20° point to the left of the crown, gives practically both positive and negative maxima. The effect of assuming this live

load to exert horizontal pressure on the arch is shown in Curve 3, being merely a shifting of the curve to the right with a slight increase in the maximum negative. While the above live load probably gives the most unfavorable working condition, Curve 4 indicates that

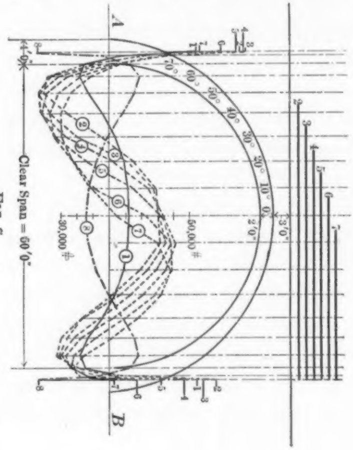

FIG. 7

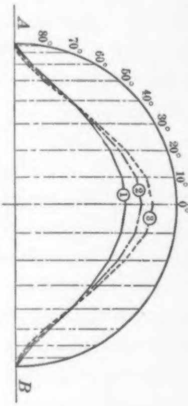

FIG. 5

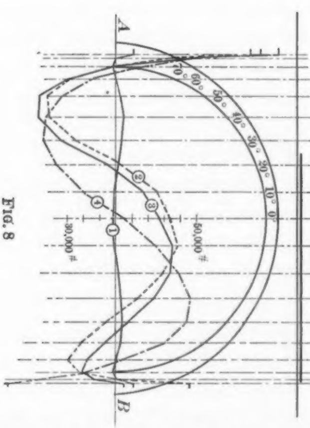

FIG. 8

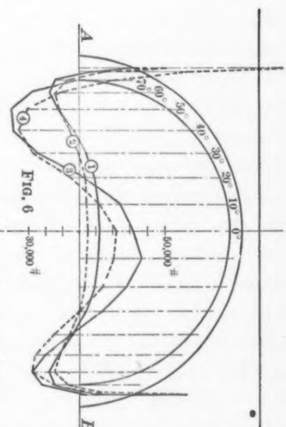

FIG. 6

erection loads must be reckoned with, showing the result of backfilling earth on the arch from the abutment, *B*, up to the crown. It is to be noted that the maximum positive as well as the abutment moments are all greater than those produced by the live load.

Fiber Stresses.—Fiber stresses, being the result of the combined effect of the bending moment and the thrust, and varying with the dimensions of the arch ring, it will be necessary to refer again to the example in order to trace the influence of a change in the ring thickness toward the abutment.

For this purpose it will be sufficient to consider the material homogeneous and the modulus constant within the range of stress, so that the ordinary formulas will apply.

The average pressure (Table 5) is indicated in the column headed "Average pressure per square inch," and the maximum fiber stress in tension and compression in the columns headed p_1 and p_2. Case I refers to the assumption of constant thickness, Case II to the abutment one and one-half times the thickness at the crown, and Case III to the abutment twice the thickness at the crown.

TABLE 5.—UNIT STRESSES—VERTICAL LOADS ONLY.

CROWN.

	Total M, in foot-pounds.	Total T, in pounds.	$\frac{M}{H}$	Average pressure, in pounds per square inch.	p_1	p_2
Case I	45 447	25 487	21.36 in.	88	+ 558	− 382
Case II..........	37 418	27 158	16.56 "	94	+ 483	− 295
Case III..........	27 931	28 381	11.76 "	99	+ 390	− 192

40° POINT.

	Total M.	Total H.	Total T.	$\frac{M}{T}$	Average pressure, in pounds per square inch.	p_1	p_2
Case I....	45 447	25 487	38 524	14.16 in.	112	+ 444	− 220
Case II...	46 110	27 158	39 678	13.94 "	115	+ 450	− 220
Case III..	44 467	28 381	40 416	13.20 "	117	+ 440	− 206

SPRINGING.

	Total M.	Total T.	$\frac{M}{T}$	Average pressure, in pounds per square inch.	p_1	p_2
Case I..........	138 648	49 110	33.88 in.	85	+ 442	− 275
Case II..........	169 521	48 892	41.61 "	85	+ 527	− 357
Case III..........	198 644	48 710	48.94 "	85	+ 605	− 435

Apparently, Case I gives stresses which are too large at the crown and too small at the abutment, while, at the 40° point, there is little variation for the three cases. To include the effect of the horizontal earth pressure would reduce the stresses and materially increase the stability of the arch, as will be seen from the following table:

UNIT STRESSES DUE TO VERTICAL DEAD AND LIVE LOAD AND HORIZONTAL DEAD LOAD.

	Total M, in foot-pounds.	Total T, in pounds.	$\frac{M}{T}$	Average pressure, in pounds per square inch.	p_1	p_2
Crown	+ 30 925	29 733	12.48 in.	103	+ 420	− 216
40°	36 861	41 136	10.75 "	120	+ 389	− 149
Springing	95 029	49 110	23.22 "	85	+ 332	− 161

The advantage of reinforcement in semicircular arches is very clear since the determining factor in the design is not the compressive but the tensile stress.

Summary.—The general effect of increasing the thickness of the arch ring toward the abutment is equivalent to shortening the span, that is, decreasing the bending moments except at the abutment, and increasing the horizontal thrust.

If the horizontal earth pressure is equal to one-third of the vertical, the bending moments due to vertical and horizontal earth loads will practically balance each other.

The bending moments being large, compared with the thrust, economy of material is secured by taking care of the tensile stress with reinforcement rather than by increasing the ring thickness.

Bending moments of either sign may occur throughout the span, making a single line of reinforcement impractical.

Back-filling the arch may cause greater fiber stresses than the live load, and should be given consideration in the design.

The author is deeply indebted to Mr. A. P. Clark for assistance in carrying out the tedious computations involved in making the tables, also for many valuable suggestions.

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TRANSACTIONS

Paper No. 1087

THE EFFECT OF TEMPERATURE CHANGES ON MASONRY.*

By Charles S. Gowen, M. Am. Soc. C. E.

With Discussion by Messrs. George G. Honness, Thaddeus Merriman, William Lowe Brown, and Charles S. Gowen.

The very great increase in the use of Portland cement during the past fifteen years has resulted in the design and building of structures of dimensions which would not have been deemed practicable at an earlier period. The general use of Portland cement concrete, with or without steel rods, or in connection with heavy stones to form "Cyclopean" masonry, is familiar to all. As a result, more compact, denser and stronger masonry is being built, particularly in hydraulic construction work, than ever before. The effect of temperature changes upon masonry walls built along such or similar lines has been noticeable in many structures, and, of course, has caused investigation by numerous engineers. It is obvious that the larger and more exposed the structure, and the denser and more compact the masonry, and the stronger the cement of which the mortar is made, the greater will be the likelihood of rupture and other noticeable results from temperature changes. The common result of such changes is shown in

* Presented at the meeting of May 20th, 1908.

cracks, of greater or less size and extent, according to circumstances. The writer has some data derived from the observation of such cracks, which may be of interest.

Such results, in the writer's opinion, are chiefly due to the variations in temperature of the outside air. However, it is a well-established fact that Portland cement mortar, in setting, is likely to cause increase in the internal temperature of masonry structures. Heavy masses of such masonry are peculiarly subject to such influences, which may continue for a long time, though lessening in degree, until the final set of the mortar has taken place. Laboratory tests show activity on the part of such mortars for four or five years at least, and the mortar used in construction must be subject to similar laws of change.

There are, moreover, the changes in such masonry due to the gradual drying out of the moisture in the work, and the tendency to shrinkage as setting continues in the open air. This element of change cannot be ignored in any discussions or conclusions which investigations regarding temperature changes simply may seem to warrant, although it may be maintained that, in the case of high-breaking and quick-setting Portland cement, the larger part of the moisture not required for the purposes of crystallization is driven out of the masonry in the first stages of setting, and, therefore, is not an important factor in the results that follow during the aging of the masonry. An extended set of observations with thermophones might throw some light on these points, but the writer has not at his command any reliable records of this kind to cite. It is extremely desirable that such information, if it exists, should be made generally available.

During the construction of the New Croton Dam, various cracks in the masonry sections occurred, and in some cases means were taken to measure their widths and variations with the changing seasons. These observations were carried on as long as circumstances permitted, and, in the case of two cracks in particular, one of which developed in the top of the finished overflow section, the observations lasted for about two years.

The profile, Plate XLVII, shows the masonry outline of the New Croton Dam, in which the cracks observed were located. These cracks were first noted for investigation in the fall and early winter of 1901-02, and brass bolts were placed, one on each side of the cracks in question,

PLATE XLVII.
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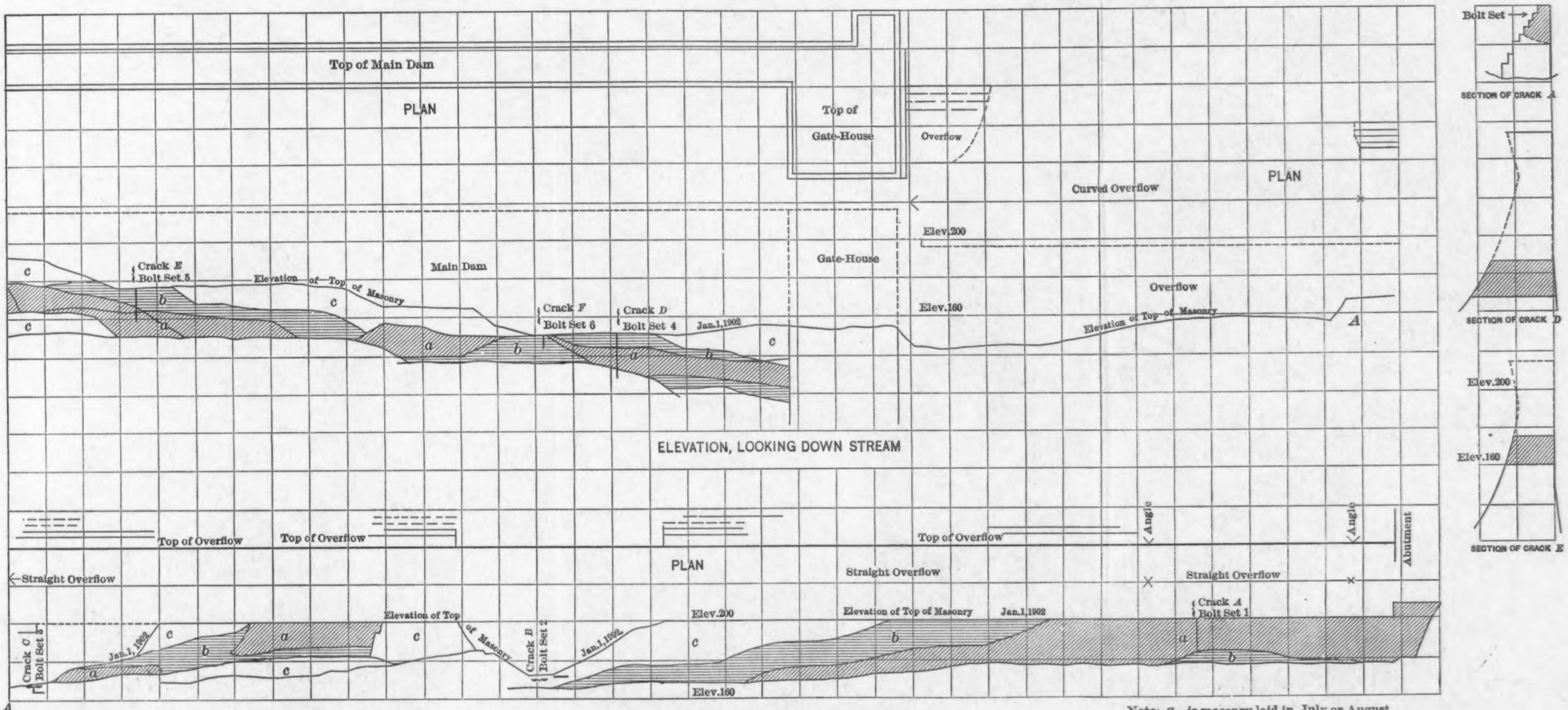
GOWEN ON
EFFECT OF TEMPERATURE CHANGES ON MASONRY.



Note: *a* is masonry laid in July or August.
b " " " " June or September.
c " " " " other months.

and about 11 in. apart. The tops of the bolts were carefully surfaced and marked, and readings of the distances between the marks were made with a scale graduated to sixtieths of inches. By using a reading-glass, these readings were estimated to $\frac{1}{600}$ in.

The profile shows the location of these cracks and their designations, as follows:

Crack	A,	Bolt Set	1
"	B,	" "	2
"	C,	" "	3
"	D,	" "	4
"	E,	" "	5
"	F,	" "	6

The diagram, Plate XLVIII, shows the observed readings plotted for Bolt Sets 1, 2, 3, and 5. The readings for Sets 4 and 6 were not plotted, as these were observed only from January to April, 1902. It will be noted that the readings on Sets 1 and 2 extend over a period of nearly two years. The other observations were of much shorter duration.

Near Bolt Set 1 a third bolt was placed in the masonry 76 ft. away. Between this and the nearest bolt of the set there was no crack in the masonry, if the hair cracks showing in the close masonry jointing of the cut stones are excepted. These developed as the joint mortar gradually set, and more particularly in the joints of the upper or coping course, which was very heavy, the stones averaging 7 ft. in length, 2½ ft. in rise and 3 ft. in width, in this case the width being assumed as the dimension in the direction of the overflow length.

A bolt was similarly placed 48 ft. away from Bolt Set 4 and one about 100 ft. away from Set 5. It was planned to measure these lengths of masonry at frequent intervals in order to compare the results with the curve of mean atmospheric temperature. The results of these measurements and the temperature curve are also shown on Plate XLVIII, from which are derived Tables 1, 2, and 3.

As Bolt Set 2 was placed to measure the possible motion of a stone creeping on its bed, or the motion resulting from a horizontal crack at the foot of a rack; and as Set 3 was placed to measure the motion or change due to a combined vertical and horizontal crack near the foot of a rack, the differences as compared with each other and with the results for Sets 1, 4, 5, and 6 are interesting.

TABLE 1.—EXTREME DIFFERENCES IN WIDTHS OF CRACKS OBSERVED, AND THE EXTREME DIFFERENCES IN OBSERVED TEMPERATURES.

Numbers of bolt sets.	Date.		Observed temperature, in degrees, Fahrenheit.	Scale readings on bolts, in sixtieths of an inch.	Differences in temperature, in degrees, Fahrenheit.	Differences in Scale Readings on Bolts: $\frac{1}{60}$ in.	Differences in Scale Readings on Bolts: Inches.	Difference in scale readings on bolts per degree of difference in temperature, in inches.
1	Jan. 16th,	1902...........	20	479.0	43	5.5	0.0917	0.00213
	Sept. 2d,	1902...........	63	473.5	50	4.9	0.0817	0.00163
	Feb. 20th,	1903...........	13	478.4	62	4.6	0.0767	0.00124
	July 30th,	1903...........	25	473.8	73	5.9	0.0983	0.00135
	Jan. 6th,	1904...........	2	479.7				
						Sum.	0.3484	0.00635
						Mean.	0.0871	0.00159
2	Feb. 8th,	1902...........	12	258	58	2.0	0.0333	0.00057
	July 10th,	1902...........	70	256				
	Aug. 5th,	1902...........	73	256	61	1.6	0.0267	0.00040
	Feb. 20th,	1903...........	12	257.6	63	1.7	0.0283	0.00045
	July 30th,	1903...........	75	255.9	75	1.9	0.0317	0.00040
	Jan. 5th,	1904...........	0	257.8				
						Sum.	0.1200	0.00182
						Mean.	0.0300	0.00045
3	Feb. 5th–15th,	1902...........	17	427.2	53	3.2	0.0533	0.00100
	June 3d–7th,	1902...........	70	424.0				
5	Feb. 11th–24th,	1902...........	20	477.0	43	4.2	0.0700	0.00163
	May 8th,	1902...........	63	472.8				

The maximum variation in the width of the cracks, as shown by the measurements of Set 1, was 0.0983 in., or about $\frac{1}{10}$ in.

Reference to Plate XLVIII will show how closely the changes in these crack curves, 1, 2, 3, and 5, follow the curve of mean temperature.

This curve of mean temperature is derived from mean daily temperatures from four observations per day, including the maximum and minimum readings of the thermometer on each day.

TABLE 2.—EXTREME DIFFERENCES IN THE WIDTHS OF THE CRACKS OBSERVED, AND THE OBSERVED TEMPERATURES, AS BEING THE MEAN IN CERTAIN CASES OF SEVERAL DAYS' OBSERVATION.

Numbers of bolt sets.	Date.	Observed temperature, in degrees, Fahrenheit.	Scale readings on bolts, in sixtieths of an inch.	Differences in temperature, in degrees, Fahrenheit.	DIFFERENCES IN SCALE READINGS ON BOLTS:		Differences in scale readings on bolts per degree of difference in temperature, in inches.
					$\frac{1}{60}$ in.	Inches.	
1	Jan. 14th-18th, 1902..........	20	479.0	43	5.5	0.0917	0.00213
	Sept. 2d, 1902..........	63	473.5	53	4.9	0.0817	0.00154
	Feb. 20th 1903..........	10	478.4	60	4.6	0.0767	0.00128
	July 14th-Aug. 14th, '03......	70	473.8	70	5.9	0.0983	0.00140
	Jan. 6th, 1904..........	0	479.7				
					Sum.	0.3484	0.00635
					Mean.	0.0871	0.00159
2	Feb. 4th-8th, 1902..........	12	258	58	2.0	0.0333	0.00057
	July 1st-12th, 1902..........	70	256				
	Aug. 5th, 1902..........	73	256	63	1.6	0.0267	0.00042
	Feb. 20th, 1903..........	10	257.6	55	1.7	0.0283	0.00052
	July 15th-Aug. 5th, '03.......	65	255.9	65	1.9	0.0317	0.00049
	Jan. 5th, 1904..........	0	257.8				
					Sum.	0.1200	0.00200
					Mean.	0.0300	0.00050
6	Feb. 8th-18th, 1902...........	20		24	1.9	0.0317	0.00132
	Apr. 16th, 1902...........	44					
4	Jan. 18th, 1902...............	22		22	2.4	0.0400	0.00182
	Apr. 16th, 1902...............	44					

Plate XLVIII has, also, in connection with the curve of mean temperature, a curve showing the measured length of the mass of masonry adjoining Bolt Set 1, about 76 ft. long. This curve is intended to show the variations in length of this masonry mass due to changes of temperature. The various lengths were obtained as often as possible—about once a week—and the observations cover more than two years. Lengths were obtained by careful measurements with a steel tape, read to $\frac{1}{1000}$ ft., under a uniform strain or stretch,

TABLE 3.—MOVEMENTS OR VARIATIONS IN WIDTHS OF CRACKS PER DEGREE OF TEMPERATURE CHANGE, BASED UPON THE EXTREME DIFFERENCES OBSERVED IN TABLES 1 AND 2.

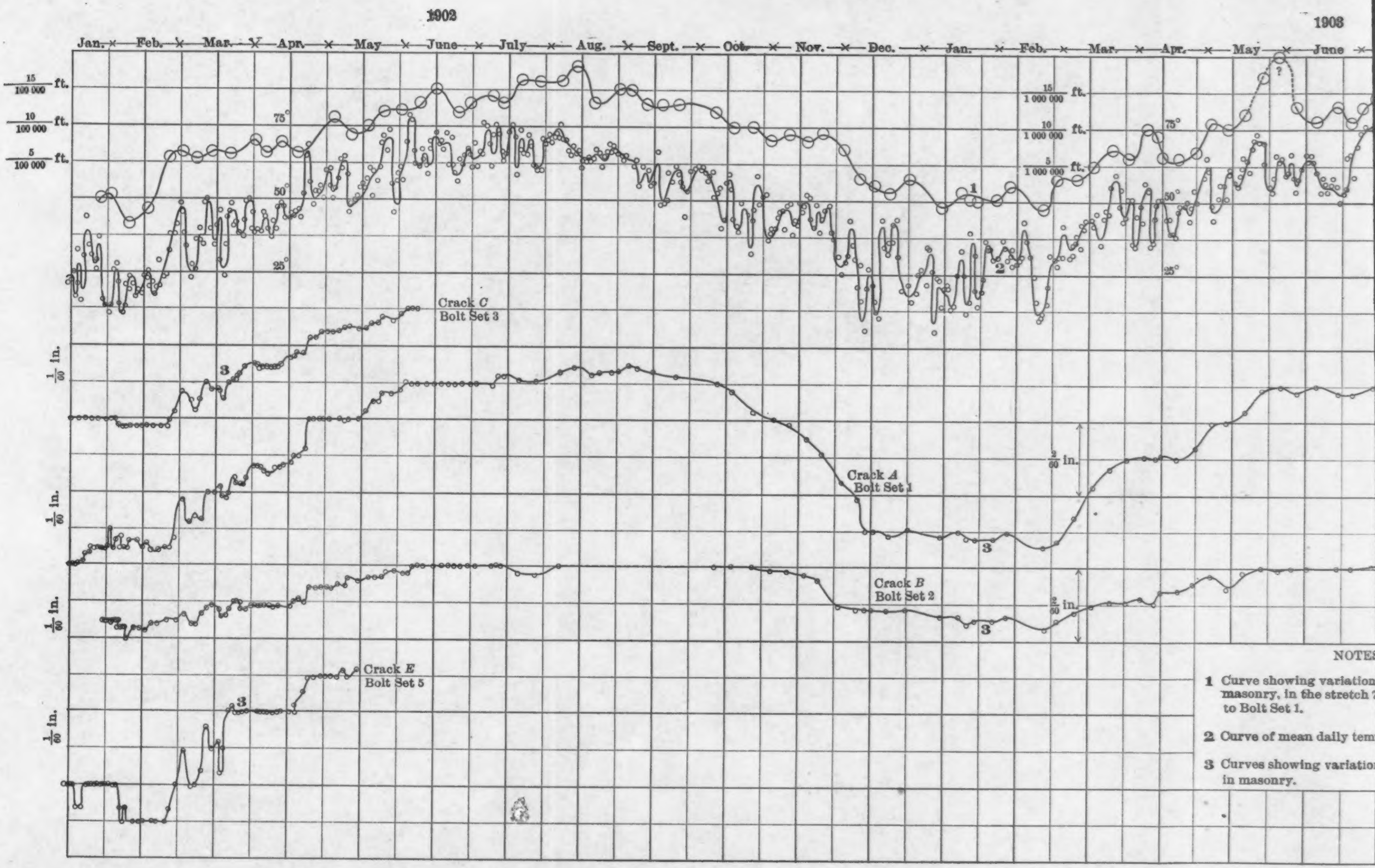
Number of Bolt Set.	Inches per degree, Fahrenheit.
1	0.00159
6	0.00132
4	0.00182
5	0.00163
Sum	0.00636
Mean	0.00159
2	0.00050
3	0.00100

and corrected for its temperature, assumed to be that of the air at the time the measurement was taken. These observed lengths were reduced to the lengths due to a temperature of 50° fahr., and the differences in measured lengths between the bolts resulting are assumed to be due to the varying length of the masonry mass. On Plate XLVIII the ordinates show these differences in length per foot of masonry measured, from time to time of measurement, reduced and plotted to $\frac{1}{1\,000\,000}$ ft.

It will be noted that this curve or diagram follows the temperature curve closely, and, as might be expected, shows a varying length of the masonry mass due to temperature changes. This length of masonry measured was the crest of the overflow, the bolts being placed on the second course, or first step down (see point marked Bolt Set 1 on Plate XLVII), and is composed of very heavy blocks of first-class masonry, as previously described, with close joints and some small proportion of backing of rubble masonry in mortar.

To what extent the enlarging section below the level of measurement, with its increasing proportion of rubble masonry backing, may have affected these changes is, of course, problematical.

It is realized that such a mode of measuring a mass of masonry is crude, but it was the only available way, and dependence is placed upon the number of observations and the consistency shown in the results, rather than in any very great degree of accuracy in the method followed, although such accuracy would have been desirable.


1902
1903
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.
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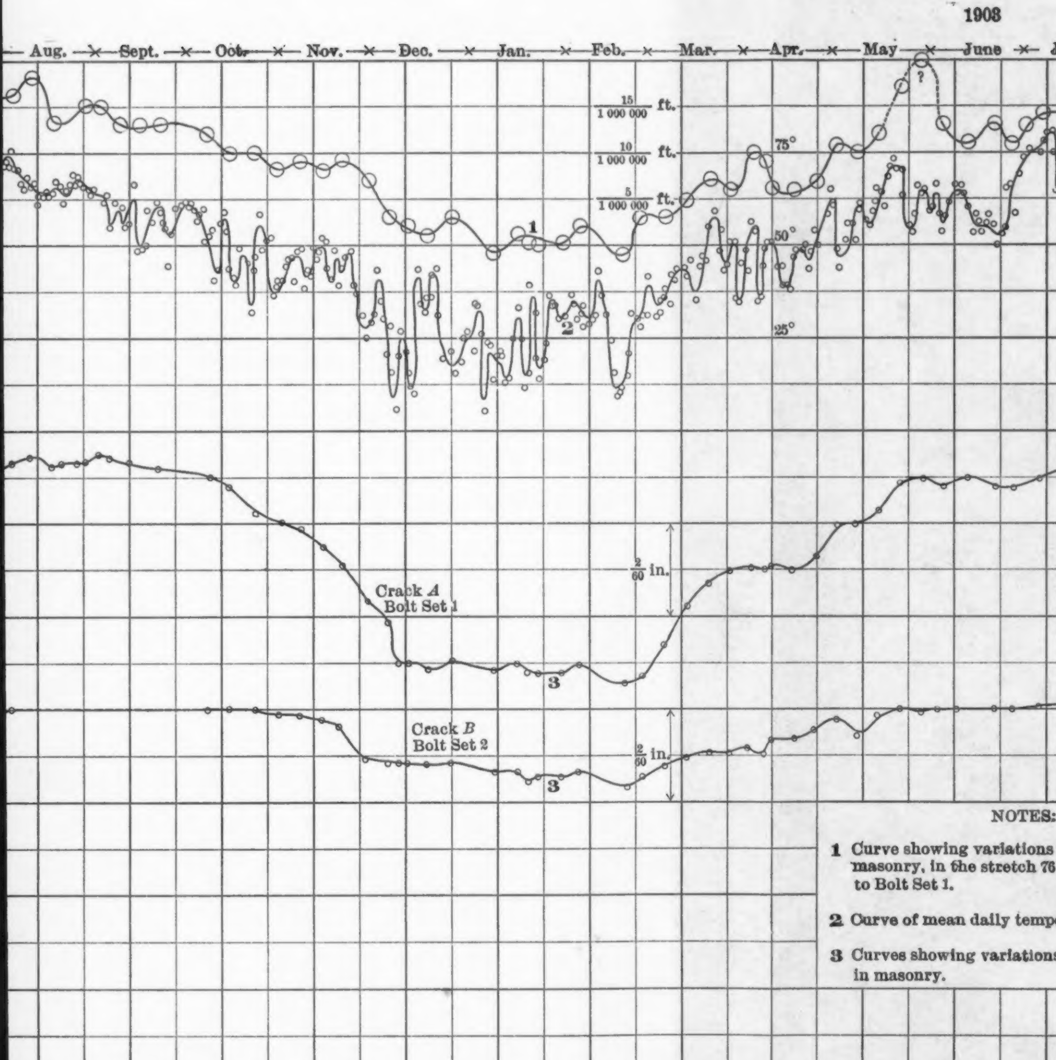
Mar.
Apr.
May
June
15/100 000 ft.
10/100 000 ft.
5/100 000 ft.
15/1 000 000 ft.
10/1 000 000 ft.
5/1 000 000 ft.
75°
50°
25°
1
2
3
?
Crack C
Bolt Set 3
Crack A
Bolt Set 1
Crack B
Bolt Set 2
Crack E
Bolt Set 5
1/60 in.
2/60 in.
NOTES
1 Curve showing variation masonry, in the stretch to Bolt Set 1.
2 Curve of mean daily tem
3 Curves showing variation in masonry.



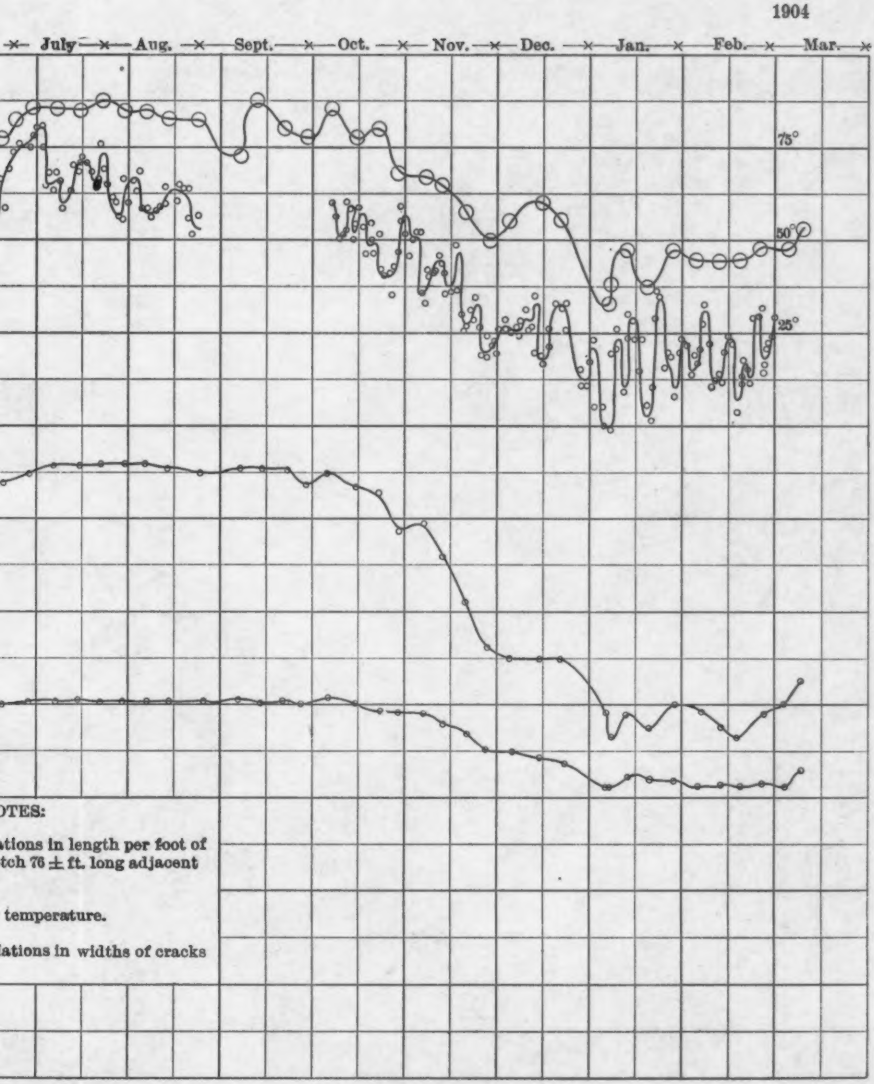
NOTES:

1 Curve showing variations
masonry, in the stretch 76
to Bolt Set 1.

2 Curve of mean daily temp

3 Curves showing variations
in masonry.

PLATE XLVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1087.
GOWEN ON
EFFECT OF TEMPERATURE CHANGES ON MASONRY.



OTES:

ations in length per foot of
tch 76 ± ft. long adjacent

temperature.

ations in widths of cracks

Table 4 shows the extreme variations in length per foot of measured mass, in comparison with temperature variations as shown in the curve.

TABLE 4.

Date.	Variation, in feet.	Temperature, in degrees.	Difference between extreme variations of temperature and length:		Variations in length per foot per degree, in feet.
			Degrees.	Feet.	
Feb. 8th, 1902....	—0.000035	12			
			61	0.000215	0.00000352
Aug. 11th, 1902..	+0.000180	73			
			61	0.000190	0.00000311
Feb. 20th, 1903...	—0.000010	12			
			61	0.000160	0.00000264
July 30th, 1903...	+0.000150	73			
			73	0.000220	0.00000300
Jan. 5th, 1904....	—0.000070	0			
				Sum.	0.00001227
				Mean.	0.00000307 per foot per degree.

The result shown in Table 4 indicates that, in the case of massive cut-stone or dimension-stone masonry, influenced to an indeterminate extent by a backing of rubble, all very compactly laid with Portland cement mortar, the rate of expansion is about half of that generally accredited to steel. To what extent the figured change of length on which this result is based may be due to the gradual drying out of the masonry as it grew older and therefore less sensitive to changes of moisture is, of course, a question. Possibly the higher coefficients (the first two in Table 4) may be due in a measure to this.

That such a coefficient derived by such means would be influenced also by the varying interior temperature of the masonry due to its gradual attainment of a final set is also evident. Unfortunately, no observations bearing upon this point are available. A number of thermophones were installed at various times in the dam masonry, but the results obtained from them were not reliable.

While the results deduced above may not be conclusive, in so far as indicating a coefficient of expansion for massive masonry, the writer submits them as approximate and not illogical in their derivation. The variations in the crack widths and their correspondence with the temperature changes are sufficiently conclusive, however, as to the immediate influence that comparatively small variations in temperatures have on masonry walls; and the deduced coefficient of ex-

pansion, when considered in relation to the variations in the widths of the measured cracks, does not seem to be inconsistent. The experience at the New Croton Dam indicates that in that climate the range in temperature is not sufficiently great to cause visible cracks in masonry masses more than 40 ft. thick and less than 40 ft. long, and experience shows, further, that in thinner masses cracks are not likely to occur at intervals of less than 40 ft. Whether the "pull" developed in a mass of solid compact masonry less than 40 ft. thick and less than 40 ft. long, due to a maximum change of temperature, is insufficient to change the actual length of the structure, or whether, in other words, the mass continues rigid under the internal strains developed, may be a question, but it seems evident that a length of narrow wall 76 ft. long did vary in actual length in close accordance with temperature changes, and that these variations produced no visible cracks in the mass. As to the other masses, 48 and 100 ft. in length, respectively, which were of rubble masonry in mortar and which it was proposed to measure in a way similar to that used for the 76-ft. length, circumstances did not allow a continued series of measurements, and those obtained were too few to afford any conclusive results.

Homer A. Reid, Assoc. M. Am. Soc. C. E., in his recently published book on reinforced concrete,* gives several coefficients of expansion of stone, mortar, concrete, etc., and these have been collated in Table 5.

TABLE 5.—COEFFICIENTS OF EXPANSION OF STONE, MORTAR, CONCRETE, ETC.

Authority.	Material.	Coefficient.
Bonniceau, *Annales des Ponts et Chaussées.*	Neat Portland cement	0.00000594
	Mortar (1:2)	0.00000655
	Concrete	0.00000795
Christophe, *Le Béton Armé*	Concrete	0.00000667 to 0.00000805
Sir A. R. Binnie, *Min. of Proc.*, Inst. C. E	Concrete (1:4) beam 100 ft. long and 1 ft. square	0.000004355
W. D. Pence, M. Am. Soc. C. E., Purdue Univ	Broken-stone concrete (1:2:4) 6 × 6 × 24-in.	0.0000054
	" " " " 4 × 4 × 36-in.	0.0000056
	Gravel concrete (1:2:4) " " (1:5) in. bars 24 to 36 in. long	0.0000054
Professor Hallock, Columbia Univ	Concrete (1:2:5) (1:2)	0.00000606
Professor Dana	Granite "Aberdeen"	0.0000044
	" Native	0.0000048
	Limestone	0.0000057

* "Concrete and Reinforced Concrete Construction."

Myron S. Falk, Assoc. M. Am. Soc. C. E., in his work on concrete,* gives, in addition to some of the data in Table 5, the following: By Professor Lyford, of the Worcester Polytechnic, for concrete bars, 0.0000056 and 0.0000064.

It would seem that Sir A. R. Binnie's results may be considered the most reliable of the foregoing, as far as concrete is concerned, as they were made on by far the largest scale, and, in addition, extended over a period of two years, during which constant observations of the changes were made. It must be borne in mind that the coefficient, 0.0000030, derived by the writer, is for heavy granite masonry with a minimum of joint space and surface, while Professor Dana gives for granite 0.0000044 and 0.0000048. A mass of jointed masonry showing many hair cracks due to shrinkage might be expected to have a smaller coefficient of expansion than the granite of which it is mainly composed.

The following descriptions of the cracks measured will be of interest:

A.—This crack developed in the winter of 1900-01, the masonry having been built and finished in the season of 1900. It is a vertical crack in the joints between the coping and facing stones on both up-stream and down-stream faces of the overflow, and extends from 14 to 17 ft. down from the top of the coping. In its length is a facing stone on the down-stream side or face cracked or ruptured vertically.

B.—This crack developed at the same time as *A.* It is at the foot of a rack left for a temporary opening in the masonry, and shows on the up-stream face as a fine horizontal crack in the bed joint of the lower facing stone of the rack. Below this course lay a continuous course of facing stones laid five years before the masonry above. This horizontal crack was about 4 ft. long on the up-stream face and could be traced as a hair crack across the dam section under the lowest hearting course of the rack to the down-stream face.

The brass bolt, Set 2, was placed in the continuous course of the up-stream facing at this point, within 2 in. of the end of the facing stone under which the crack was noted.

C.—This crack developed in the winter of 1901-02. It shows on the up-stream side of the dam as a vertical crack in the facing stone

* "Cements, Mortars and Concretes."

joints extending down two courses, about 4 ft., and then horizontally under the second course for about 5 ft. It was traced across the dam section as a hair crack under the lower course of the hearting stone of the rack, losing itself in the joints of the down-stream facing stone, where, however, a knife could be run into the joint. Later, the course of stones on the face of the rack under which the crack showed was taken out, and it was found that then the crack pinched out, no further trace being visible. Two brass bolts (Set 3), one on each side of this crack on the up-stream side, were placed in February, 1902.

D.—This crack developed in the winter of 1901-02. It extended down 20 ft. on the up-stream side vertically following the joints between the facing stones, except that one stone, two courses down, was found cracked. On the down-stream side the crack could be traced as a hair crack in the joints for nearly the same distance down vertically. The crack, of course, extended across the top of the masonry. Two brass bolts (Set 4) were placed, one on each side of the crack on the top of the up-stream facing course. The hearting of the dam on either side of this crack was dug out in June, 1902, to a depth of 13 ft. The crack was still traced, however, at the bottom of this hole, as crossing the dam section and showing in the mortar joints. The hole was tested by filling it with water before the masonry was rebuilt, and a slight seepage was noticeable on the up-stream face of the dam, 20 ft. down.

E.—This crack developed in the winter of 1901-02. It is a vertical crack extending across the dam and down about 18 ft. on both up- and down-stream faces, showing two cracked facing stones in each face. Two brass bolts (Set 5) were placed, one on each side of this crack on top of the up-stream facing course. The crack was dug out to a depth of 4 ft., but showed no traces in the hearting masonry at this depth. It was dug out in June, 1902, and the hole held water when filled.

F.—This crack developed in the winter of 1901-02. It is a vertical crack extending across the dam and down 5 or 6 ft. on both faces, in the joints of the facing stone. A brass bolt (Set 6) was placed at this crack.

The lessons taught by these and other cracks may be summarized as follows:

The more compact the masonry the greater the likelihood of cracks due to changes in temperature.

The localization and extent of such cracks are due primarily to the differences occurring in sectional areas. These arise from carrying on the masonry work in racks or other unequal elevations of progress in the masonry, or from changes in section due, for example, in the case of dams, to buttresses, gate-houses, angles, etc., and they develop along straight lines. At the New Croton Dam, in the curve forming part of the overflow, no cracks were found during its construction.

If these inequalities in sectional areas result as they are bound to do in many cases, in a varying sectional mass of masonry laid in warm weather or summer and particularly if this is unprotected by masonry laid above it later in the season, then the most pronounced cracks are likely to occur.

Temperature changes affect to the greatest extent masonry laid in warm weather, and it is important, in order to diminish the extent of temperature cracks to the minimum, to carry such masonry up in fairly level layers and to cover it with as much masonry as possible by steady progress in the colder months.

Changes in section of masonry structures are bound to produce cracks within certain limits, and their extent depends upon the amount of the sectional change and the time of year the masonry is laid. A dam or wall of uniform section and height carried up uniformly from one end to the other should show temperature cracks at regular intervals, their extent, width, and spacing depending upon the time of year in which the masonry is laid.

That cracks can be avoided to an extent by following rules derived from the above is evident. If uniformity of conditions of building, however, cannot be maintained during construction, cracks can be provided for by vertical slip joints at the proper places, extending through the wall, with proper returns or grooves to break joint, and treated with coal-tar, asphalt, or in other ways that might suggest themselves.

In conclusion the writer wishes to make suitable acknowledgments to the New York Aqueduct Commission, in whose service he was at the time the data were obtained from which the foregoing results have been derived.

DISCUSSION.

Mr. Honness.

GEORGE G. HONNESS, ASSOC. M. AM. SOC. C. E.—The speaker is glad that the subject of the effect of temperature changes on masonry has been brought before the Society, and wishes to add his observations of such effects on large masonry structures which have been under his supervision, in the hope that such information may lead to methods of caring for the resulting temperature cracks.

In addition to the more general use of Portland cement and the resulting character of masonry, it seems likely that the greater rapidity of construction attained in recent years is another factor which has led to the more frequent and noticeable development of temperature cracks, because of the probably greater range of temperature caused by the retention of the heat generated by the setting mass of masonry, which heat must, of necessity, be retained for longer periods.

As an example of the progress of laying masonry a few years ago, compared with the present practice, the following is cited:

"At the Sodom Dam, near Brewster, N. Y., in the Croton water-shed, the maximum monthly progress was 3 000 cu. yd., this being rubble masonry, laid partly in American and partly in Portland cement. At the Titicus Dam, at Purdys Station, N. Y., also in the Croton water-shed, the maximum monthly progress was 5 700 cu. yd., the character of the masonry being the same as at the Sodom Dam. At the Boonton Dam, Boonton, N. J., the maximum monthly progress was 21 000 cu. yd., and at the Cross River Dam, Katonah, N. Y., the maximum monthly progress was 18 400 cu. yd."

The two structures of hydraulic masonry which the speaker has had an opportunity to observe closely are the Boonton Dam and the Cross River Dam.

The Boonton Dam.—The Boonton Dam, at Boonton, N. J., is built of cyclopean masonry, the down-stream face being ashlar, above the lines of refilling, and the up-stream face, rubble masonry. In all, about 255 000 cu. yd. of masonry were required for this structure, Portland cement being used exclusively.

In the season of 1902, from May 20th, 84 000 cu. yd. were placed; in 1903, 134 000 cu. yd., and in 1904, 37 000 cu. yd.

No cracks in the dam were discovered in the winter of 1902-03, which is probably accounted for by the fact that the masonry laid in the season of 1902 was all placed in the lower portions of the dam section where the width was great enough to withstand the stresses due to the ranges of temperature, and, to some extent, for the additional reason that a layer of masonry was laid over the entire portion of the dam then under construction in the colder months.

In the second and third winter seasons, however, numerous vertical cracks developed. There was no regularity in the distance between

FIG. 1.—DOWN-STREAM FACE OF CROSS RIVER DAM.

FIG. 2.—UP-STREAM FACE OF CROSS RIVER DAM.

the cracks, and in no place were there any evident signs of horizontal cracking. The "racks," which seemed to be unavoidable, apparently had no influence on the location or the number of cracks. The largest cracks were near the ends of the dam, and at least six of them leaked a considerable quantity of water, which leakage materially diminished in the summer months, but did not cease entirely. Mr. Honness.

No effort of any kind was made to treat these cracks or to diminish the leakage.

By measurement and estimation, the total width of all the cracks on top of the dam was 3½ in., and the largest ones could be traced down the face of the dam for a distance of 60 ft. from the top.

Thermophones were embedded at different points in the dam section, and some interesting data regarding the interior temperatures of a large mass of masonry were obtained. Thaddeus Merriman, Assoc. M. Am. Soc. C. E., who was Principal Assistant Engineer on this work, has made an analysis of the data obtained from these thermophone records and of the temperature cracks in this dam.

Cross River Dam.—The Cross River Dam is built of cyclopean masonry and faced, both up stream and down stream, with concrete blocks. Fig. 1, Plate XLIX, is a view of the down-stream face from the south end, and Fig. 2, Plate XLIX, of the up-stream face, showing the waste-weir and gate-house.

In all, 158 000 cu. yd. of masonry were required, Portland cement being used exclusively.

The upper 30 ft. of the dam was reinforced with 1¼-in. square twisted rods, or rods of equivalent area. They were placed in six horizontal layers, laid longitudinally and hooked together at the ends by right-angled bends. Plate L shows the rods of the second layer in place, and Fig. 2, Plate LII, their location. The number and position of these rods were fixed by the Chief Engineer, Walter H. Sears, M. Am. Soc. C. E., and W. H. Burr, M. Am. Soc. C. E., of the Aqueduct Commission. It was not expected that the use of these rods would entirely eliminate the temperature cracks, but it was hoped that it would prevent their concentration in a few large cracks.

In the first season, about 87 800 cu. yd. of masonry were placed, the maximum month's work being 18 400 cu. yd. During the first season, the dam was built in approximately horizontal layers by derricks placed outside the section. Work was continued until about the middle of December, when it was suspended because of cold weather. By this time the dam had been built for a height of about 92.5 ft. above the lowest point in the foundation, the maximum bottom width being 116.3 ft. and the top width, 51 ft. No temperature cracks were discovered during the winter of 1906-07.

During the second season, 1907, the dam was built with derricks placed directly on the wall. These were arranged back to back, and built a pinnacle with a maximum height of 9 ft., the "rack" being on

Mr. Honness. a 1 to 1 slope, this being the limit of maximum height and slope allowed. Upon the completion of these pinnacles, the derricks were shifted to a similar position on the top of the pinnacle, the intervening space was filled, and another pinnacle was built thereon. This operation was continued until the dam was completed. Plate LI shows the order in which the masonry was placed, and the quantity for each month, also the manner of placing the masonry and the location of the cracks.

Work was resumed late in March, and the main dam was completed by the end of August, 1907; consequently, all masonry used in its construction during the season of 1907 was laid in the warmer weather.

The waste-weir, the section of which is about 9 ft. high and 9 ft. wide on top, was built mostly in September and October, in sections 35 ft. long, a slip-joint being provided at the end of each section.

The first temperature cracks in the main dam were noticed early in October, 1907, and were four in number, located at Stations 10 + 46, 11 + 08, 12 + 73, and 14 + 89, as shown on Plate LI. By March 1st, 1908, three of these cracks could be traced down the face of the dam for a distance of 70 ft., and the fourth for a distance of about 42 ft. from the top. There was no indication of any horizontal cracking, except at points where the cracks for short distances followed the horizontal joints of the blocks. The cracks were widest at the top, gradually diminishing in width toward the bottom. The maximum width of all the cracks was $\frac{3}{8}$ in.

No leakage or seepage through the dam or through the cracks was noticed until January 16th, 1908. On this date the water stood at a point 53 ft. below the top of the dam, and the leakage was noticed at a distance of 70 ft. from the top of the dam. Fig. 1, Plate LII, shows the crack at Station 14 + 89, which extends through a header block. A slight amount of leakage continued until February 7th, when it suddenly increased, indicating that there was a free passage for water through the dam section at the cracks. On February 10th, a measurement showed a leakage of 6.6 gal. per min., and by February 17th, this had increased to 23.9 gal. per min., which is the maximum amount of leakage measured at any time. As soon as the marked increase in leakage was noticed, steps were taken to control it. This was accomplished by caulking the crack at the up-stream face with lead wool and grouting the crack. These measures were successful to the extent of reducing the leakage to 4.5 gal. per min., and this has continued to diminish since the warm weather set in.

From these observations it would appear that the dam was entirely cracked through for a distance of about 43 ft. from the top, at which point the section is 31.75 ft. wide, and partially cracked for a distance of 27 ft. further from the top of the dam. This condition, and also the position of the reinforcement, is shown by Fig. 2, Plate LII.

PLATE L.
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ON MASONRY.

REINFORCING RODS IN THE CROSS RIVER DAM.

In the waste-weir, all slip-joints opened, but not to as great a width as the cracks in the main dam, probably due to the fact that this masonry was laid in the colder months. Mr. Honness.

From the observations made on the structures here described, the following conclusions are drawn:

1st.—While the speaker does not wish to be understood as favoring "racks" in hydraulic masonry, believing them to be objectionable, it does not appear that they, in any way, affect the number or location of cracks.

2d.—It does not appear that there is any extensive separation of the masonry on horizontal planes. The vertical cracks seem to be caused by the expansion and contraction of the molecules of the masonry itself, due to ranges of temperature.

3d.—There is apparently no uniformity in the distances between cracks. At Boonton, there is a crack for about every 100 ft. in the length of the dam, and at Cross River, one for about every 200 ft. This may or may not be due to the manner in which the masonry is placed, the irregularity in the profile of the foundation, or to the length of the dam.

4th.—The reinforcement in the Cross River Dam—as to quantity and method of placing—was not effective in preventing the concentration of the cracks. Another arrangement and a greater quantity, carried for a greater distance from the top of the dam, might prove beneficial, and distribute the effect of temperature changes in a large number of very small cracks.

5th.—It would appear that it would be best to provide for the control of the temperature cracks by the use of vertical joints, as suggested by Mr. Gowen, not overlooking, however, the fact that it is practically impossible to prevent a slight seepage, which will disfigure the face of any dam, no matter what class of facing be used. Provision should be made to prevent this seepage from reaching the down-stream face. Experience alone will show what details should be used in making these joints, but it seems certain that they will not add materially to the cost of construction.

Thaddeus Merriman, Assoc. M. Am. Soc. C. E.—This subject is of interest to every engineer. It is, moreover, one which, at the present time, is but little understood. As more facts concerning it are brought out, therefore, they will lead to a better knowledge of it and, consequently, better design of masonry structures. Mr. Merriman.

The author has presented one phase of the subject. The speaker will endeavor to present another, basing his conclusions on the observations of the interior temperatures in the masonry dam at Boonton, N. J.

The Boonton Dam was built during the years 1902, 1903, and 1904, across the Rockaway River, near Boonton, N. J., by the Jersey

Mr. Merriman. City Water Supply Company, for the purpose of furnishing a supply of water to Jersey City, N. J. Its design and construction were under the direction of Edlow Harrison, M. Am. Soc. C. E., Chief Engineer, J. Waldo Smith, M. Am. Soc. C. E., Consulting Engineer, William B. Fuller, M. Am. Soc. C. E., Resident Engineer, and George G. Honness, Assoc. M. Am. Soc. C. E., Division Engineer.

The total length of this dam is 3 100 ft., 2 150 ft. being of masonry having the cross-section shown by Fig. 2. The maximum height of the full masonry section is 114 ft., and its average height for its entire length is 90 ft. At its north end the masonry is 39 ft. in height, and at its south end the top of the dam is 67 ft. above the foundation. The axis of the dam bears 12° west of north, and the dam impounds water on its west side. The profile, Plate LIII, shows the progress on the masonry during construction, as well as the locations and sizes of the temperature cracks which have occurred. Fig. 1 indicates the number of cubic yards of masonry built during each month.

The down-stream face of the dam was faced with ashlar, as shown by Fig. 1, Plate LIV. The up-stream face was formed of rubble, as shown by the photograph, Fig. 2, Plate LIV, which also gives a view of the intake gate-house. The general character of the interior masonry is indicated by Fig. 1, Plate LV, and Fig. 2, Plate LV, shows the completed structure as water was beginning to pass over the spillway, which, it should be noted, forms a part of the main dam.

Atlas Portland cement was used exclusively in the construction of this dam. The masonry of the interior is "cyclopean," and is composed roughly of 50% of concrete and 50% of large stones. The concrete was mixed in the proportions of 1 : 3 : 6. All the stone used was a syenitic gneiss, and the sand was from local deposits, evidently of glacial origin.

The temperature cracks which have occurred are shown on the profile, Plate LIII. Those which occurred during the first winter (1903-1904) are indicated by solid lines, the upper ends of which show the height to which the masonry had advanced when work was suspended, in December, 1903. It will be observed that no cracks are shown between Stations 12 + 00 and 19 + 00. This is explainable by the fact that the upper layers of masonry between these stations were all put in during November and December of that winter. The weather during these months was extremely cold, and the surface of all the masonry then put in was frosted. The temperature range in this part of the work, therefore, was probably quite small, and the frosted condition of the surface also tended to prevent the discovery of any cracks which may have existed.

Before the winter of 1904-1905 arrived, the masonry of the dam had been completed, and the cracks shown by dotted lines made their appearance; the crack at Station 26 + 90 also again opened up.

PLATE LI.
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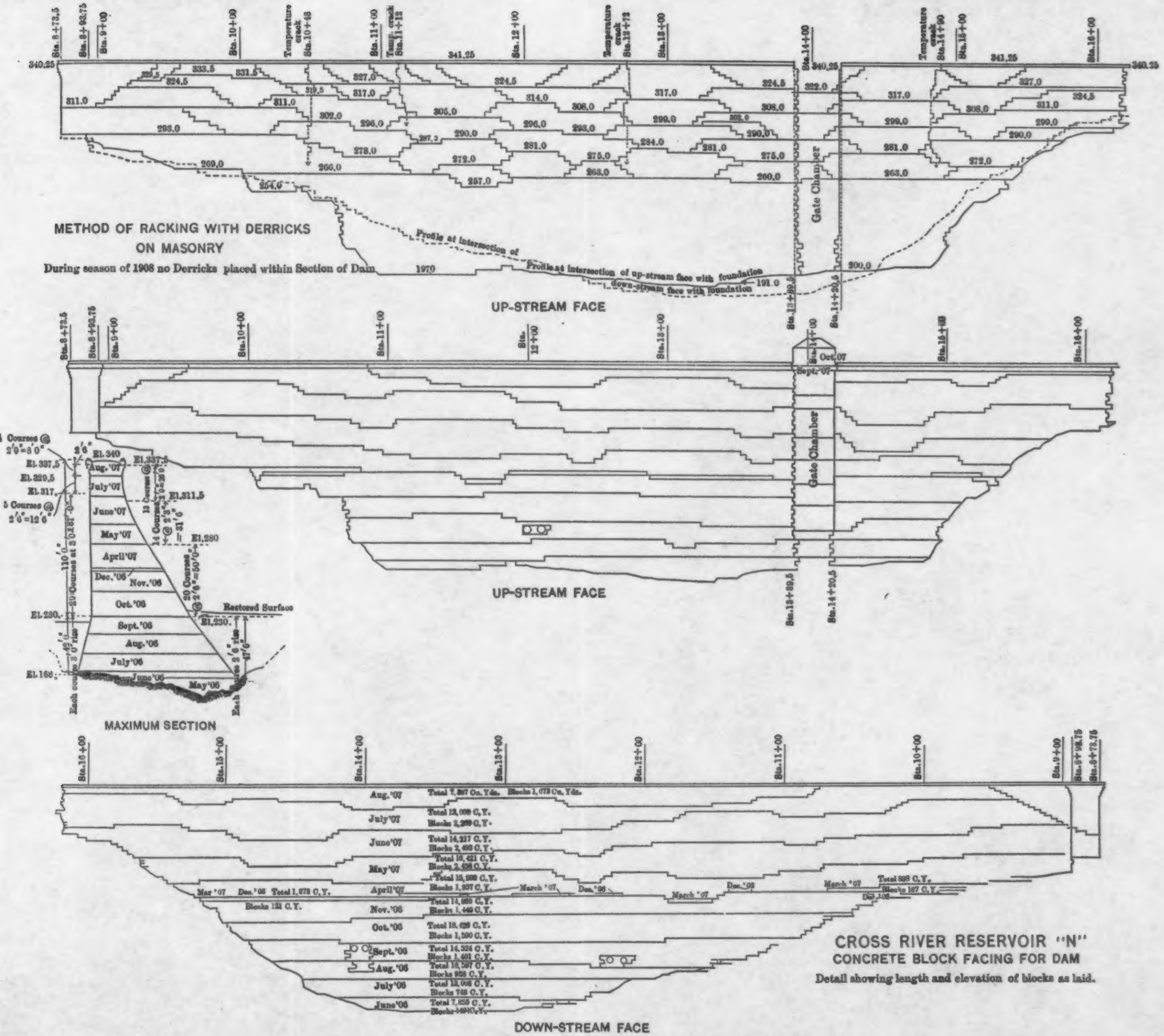

CROSS RIVER RESERVOIR "N"
CONCRETE BLOCK FACING FOR DAM
Detail showing length and elevation of blocks as laid.

During the third winter (1905-1906) no new cracks appeared, but all those which had formed during the second winter again opened up. A noticeable fact in connection with the re-opening of these cracks during the winter of 1905-1906 was that they did not all open up in the same ratio as during the previous winter; that is, one of the cracks was larger than a year ago, while one or both of its neighbors was smaller. Mr. Merriman.

During the fourth and fifth winters (1906-1907 and 1907-1908) all the old cracks re-opened, though, as already noted, not always in the same ratio, and no new cracking could be observed.

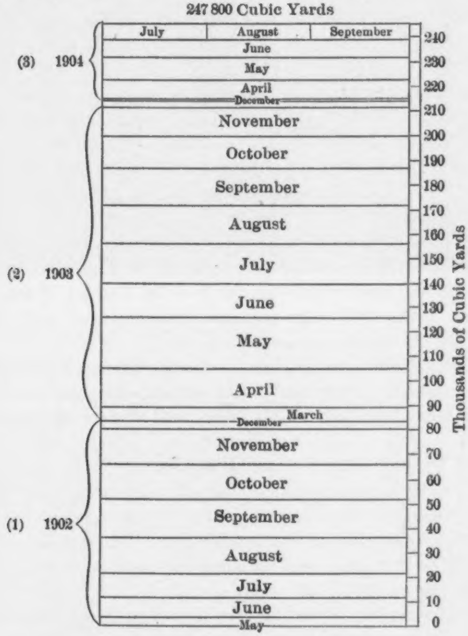

FIG. 1.

Seventeen main cracks have appeared in the masonry section of this dam, the designation, "main cracks," referring here, as elsewhere in this discussion, to cracks having a top width of $\frac{1}{16}$ in. or more. Besides these main cracks, there have appeared also 16 smaller ones, each having a top width of less than $\frac{1}{16}$ in., and, in addition to the 33 cracks which extend entirely across the top of the dam, there are about an equal number which do not extend across the section, but are confined to one-half or less of the crest width.

Mr. Merriman. **An approximate computation of the probable sum of all the cracks is as follows:**

17 cracks (dimensions on Plate LIII)		....	2.50 in.
16 "	averaging $\frac{1}{32}$ in.	....	0.50 "
33 half cracks,	" $\frac{1}{32}$ in.	....	0.50 "
	Total	....	3.50 "

A general inspection of the profile, Plate LIII, indicates that the main cracks occur at very regular intervals, and that this interval, except near the ends of the dam, is not far from 100 ft. It is also apparent that proportionately more cracks have developed in that portion of the dam in which the masonry was laid during the warmer months.

No very large quantity of water has passed through any except the main cracks. Through them, however, considerable seepage occurs. This seepage is greater in winter than in summer, and at some of them it is more or less visible during the entire year. In consequence of this seepage, the face of the dam is wet in the immediate vicinity of the crack through which it occurs, and in winter huge masses of ice are formed, so that a view of the face of the dam during the cold season presents a very interesting picture. The principal seepage occurs through the cracks at Stations 9 + 36 and 24 + 95.

On March 17th, 1908, this seepage was measured and found to be 23 000 gal. per 24 hours.

In connection with his studies of the question of the temperature changes in masonry dams and their effects, the speaker visited the New Croton Dam on March 17th, 1906, and desires to record here his observations.

The crest of the New Croton Dam serves as a roadway, and is paved with rough concrete which was put down in two layers so as to form a horizontal expansion joint. This feature rendered the determination of the sizes of the cracks more difficult than at Boonton, where the top of the dam was finished off with a granolithic surface integral with the body of the dam.

For the purpose of removing the drainage from the crest of the New Croton Dam, a tunnel, parallel with its axis, and placed in the body of the dam below its crest, was provided. This tunnel afforded good opportunity for inspecting the cracks, which, however, here as well as elsewhere, had been caulked prior to the time of the speaker's visit.

There were found in this dam five large cracks. The largest of these measured about ¼ in., while each of the other four averaged about ⅛ in. In addition to these larger cracks, there were a number of smaller ones, but just how many could not be determined, on account of the

PLATE LII.
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ON MASONRY.

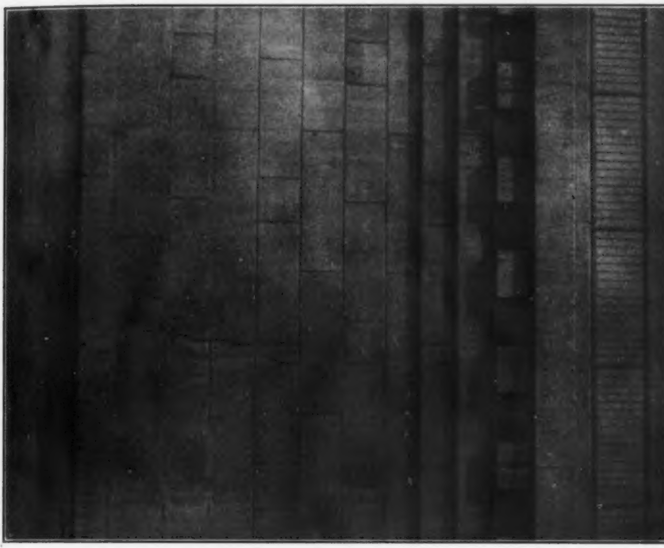


FIG. 1.—CRACK IN MASONRY OF CROSS RIVER DAM, AT STATION 14 + 89.

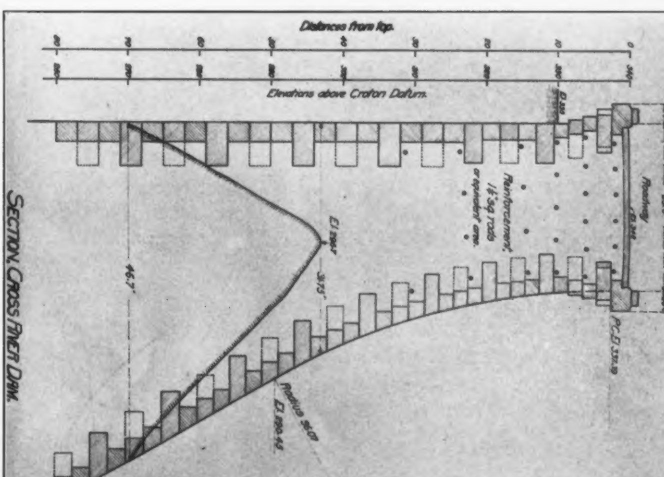


FIG. 2.—LIMIT OF CRACKS ON UP-STREAM AND DOWN-STREAM FACES OF DAM.

concrete roadway. These smaller cracks did not show in the drainage tunnel, but at the coping joints on the dam crest. Mr. Merriman.

On the spillway section of the New Croton Dam the speaker found no very large cracks, though there were ten of small size, together with a number of others showing at the joints of the coping.

The general external conditions at this dam after its completion, therefore, did not appear to differ materially from those at Boonton, and, in general, they were also similar to those described by Mr. Honness as existing at the Cross River Dam.

It is not probable that the passage of water through temperature cracks can have any appreciable influence on the life of the structure; at any rate, no more than it has on the face joints of the up-stream side; moreover, most of the evidence at hand indicates in a general way that Portland cement does not deteriorate in the presence of water, but rather, and on the contrary, that it improves both in strength and durability as it becomes older. A greater and more important question seems to be, what may be expected to be the life of a masonry dam constructed with Portland cement as now manufactured?

The passage of water through temperature cracks probably has no appreciable effect on the life of the structure, yet, by its presence on the visible face, it calls attention to the cracks themselves, and, to one unfamiliar with the technique of dam design and construction, such cracks appear only as indications of weakness, and no "explanation" can ever fully explain why such cracks should be permitted to exist.

The introduction of expansion joints, as referred to by the author, and as provided for in the plans of the Olive Bridge Dam of the Ashokan Reservoir, for the new supply from the Catskill Mountains, is a step in the right direction, though experience alone will be able to tell whether all cracking can be prevented in so simple a manner.

As work on the Boonton Dam progressed, eleven thermophones of the Whipple-Warren type were built into the masonry at Station 14 + 70 (Plate LIII). The positions of the thermophones in the cross-section are shown by Fig. 2. These thermophones were designed to be sensitive and correct to within the nearest degree. During the first year the temperatures were read directly from the graduated scale of the resistance box, the point of equalization being determined by noting the cessation of vibration in a telephonic receiver when the circuit was rapidly made and broken. Later, the telephone arrangement was replaced by a galvanometer, the temperature being read, as before, on the graduated scale.

At various times trouble was experienced with the operation of the thermophones, as indicated by the sudden fluctuations in the temperature curves on Plate LVI and Fig. 3. Then, later, the coils, one after the other, without any apparent reason, ceased recording,

Mr. Merriman. until at this time none of the eleven is giving reliable results. An investigation into the cause of the trouble soon showed that it was due to the insulation of the cables leading to the coils. The cables were of the three-strand parallel variety, the three strands being

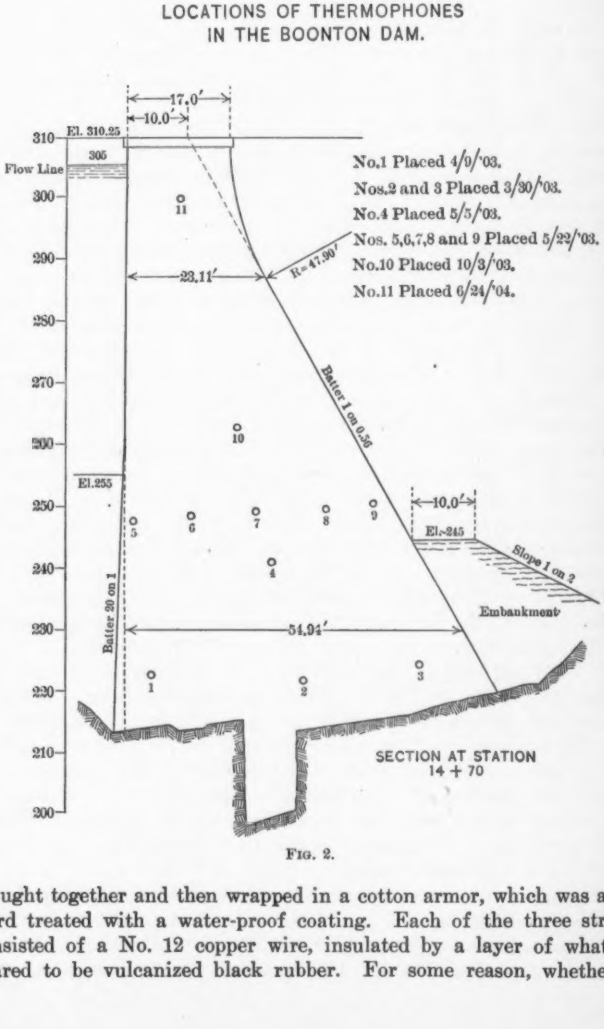
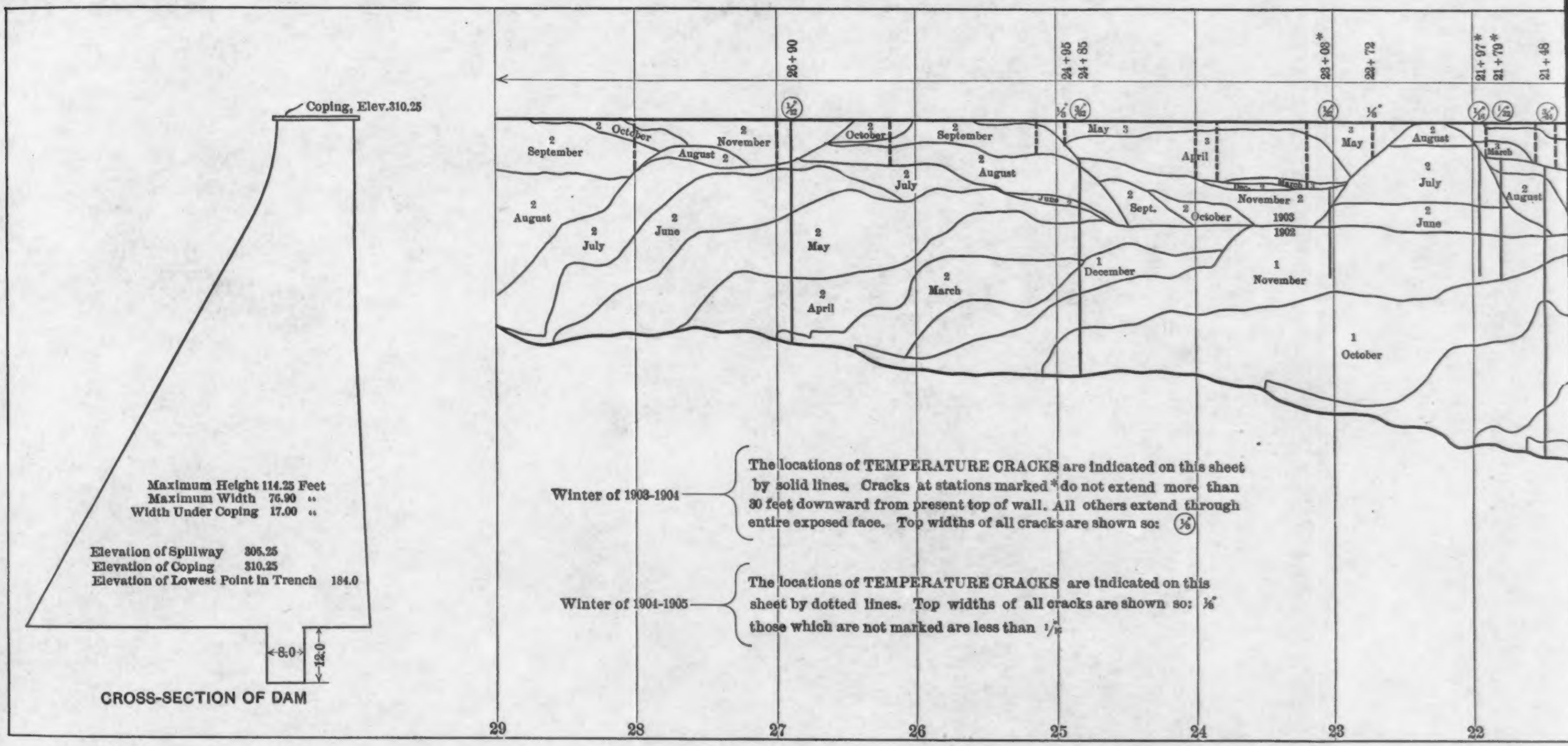
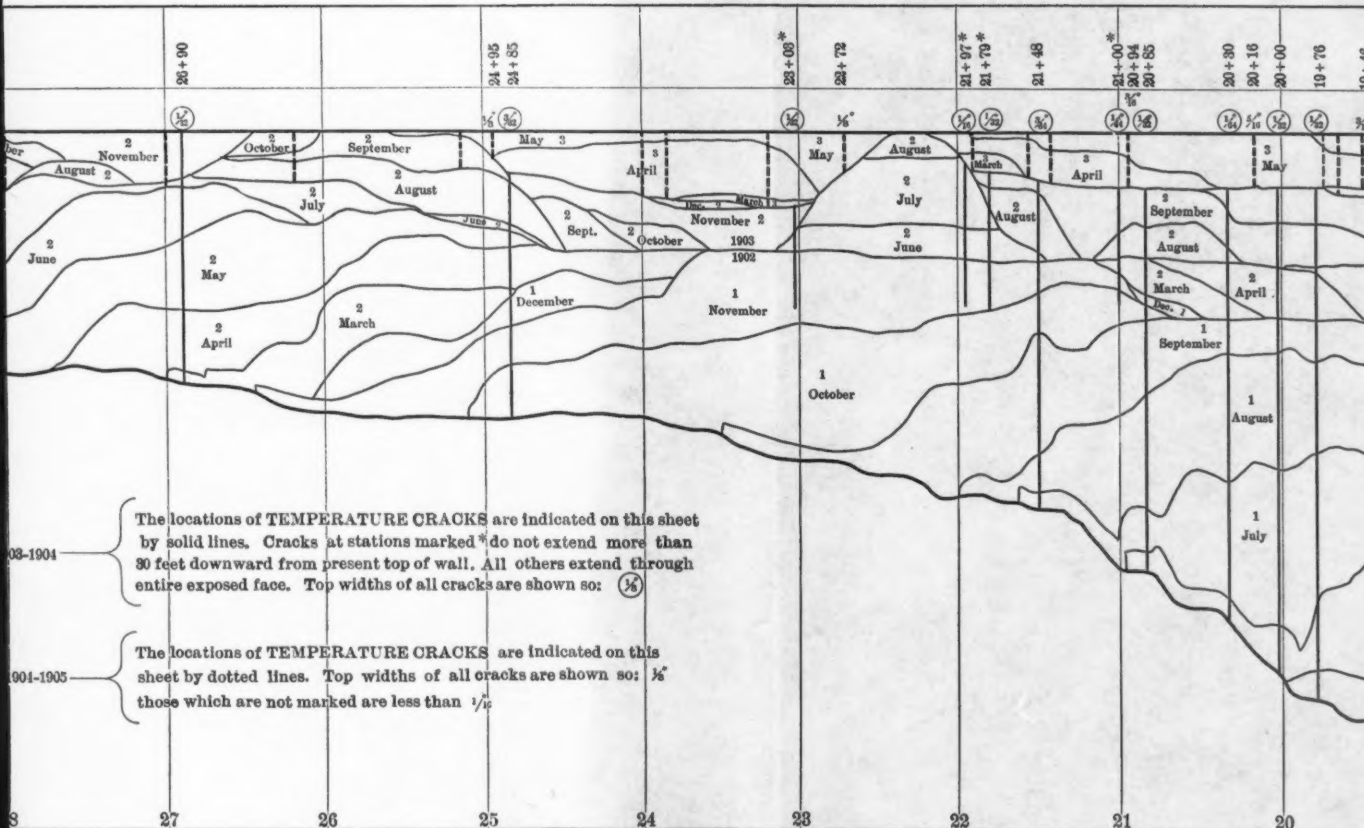


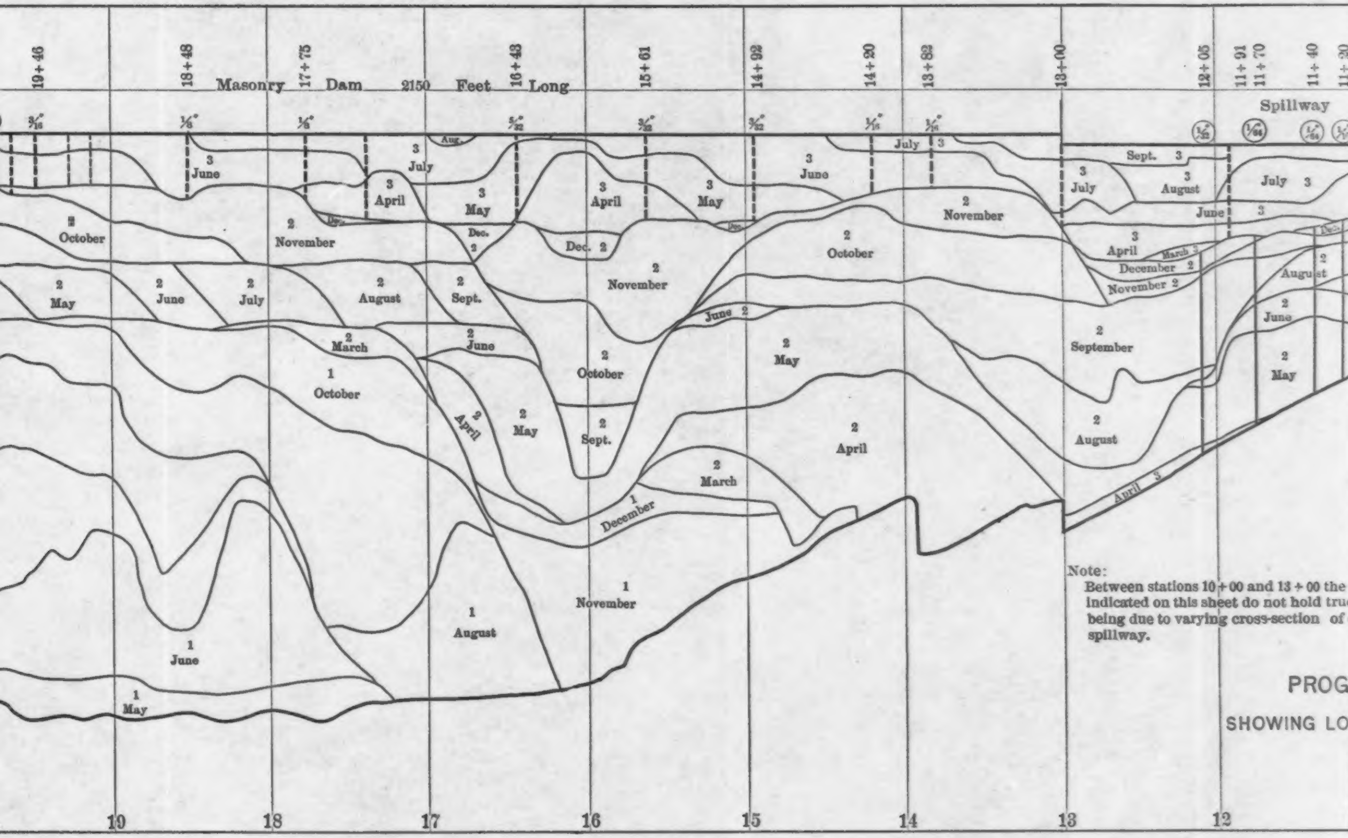
FIG. 2.

brought together and then wrapped in a cotton armor, which was afterward treated with a water-proof coating. Each of the three strands consisted of a No. 12 copper wire, insulated by a layer of what appeared to be vulcanized black rubber. For some reason, whether on

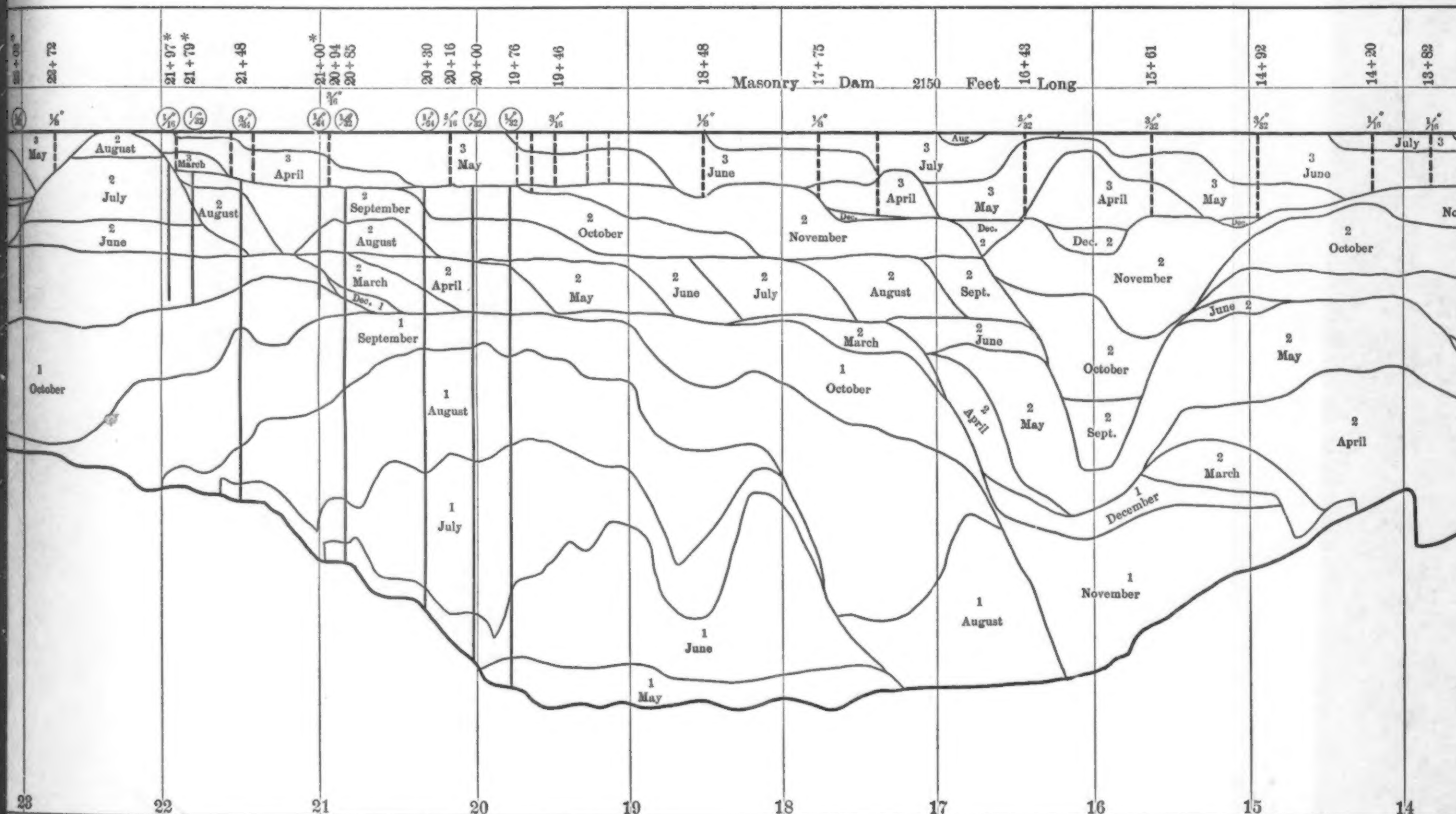

Coping, Elev. 310.25
Maximum Height 114.25 Feet
Maximum Width 76.90 "
Width Under Coping 17.00 "
Elevation of Spillway 305.25
Elevation of Coping 310.25
Elevation of Lowest Point in Trench 184.0
8.0
12.0
CROSS-SECTION OF DAM
26 + 90
24 + 95
24 + 85
23 + 08*
22 + 72
21 + 97*
21 + 79*
21 + 48
Winter of 1903-1904
The locations of TEMPERATURE CRACKS are indicated on this sheet by solid lines. Cracks at stations marked * do not extend more than 30 feet downward from present top of wall. All others extend through entire exposed face. Top widths of all cracks are shown so: 1/8
Winter of 1904-1905
The locations of TEMPERATURE CRACKS are indicated on this sheet by dotted lines. Top widths of all cracks are shown so: 1/8" those which are not marked are less than 1/16
29
28
27
26
25
24
23
22



26 + 90
24 + 95
24 + 85
23 + 08 *
22 + 72
21 + 97 *
21 + 79 *
21 + 48
21 + 00 *
20 + 94
20 + 85
20 + 30
20 + 16
20 + 00
19 + 76
19 + 46
November
August
October
September
May
April
July
August
June
Sept.
October
November
1903
1902
December
March
May
April
June
March
April
November
October
September
August
July
03-1904
The locations of TEMPERATURE CRACKS are indicated on this sheet by solid lines. Cracks at stations marked * do not extend more than 30 feet downward from present top of wall. All others extend through entire exposed face. Top widths of all cracks are shown so: 1/8
1904-1905
The locations of TEMPERATURE CRACKS are indicated on this sheet by dotted lines. Top widths of all cracks are shown so: 1/8" those which are not marked are less than 1/16
8
27
26
25
24
23
22
21
20

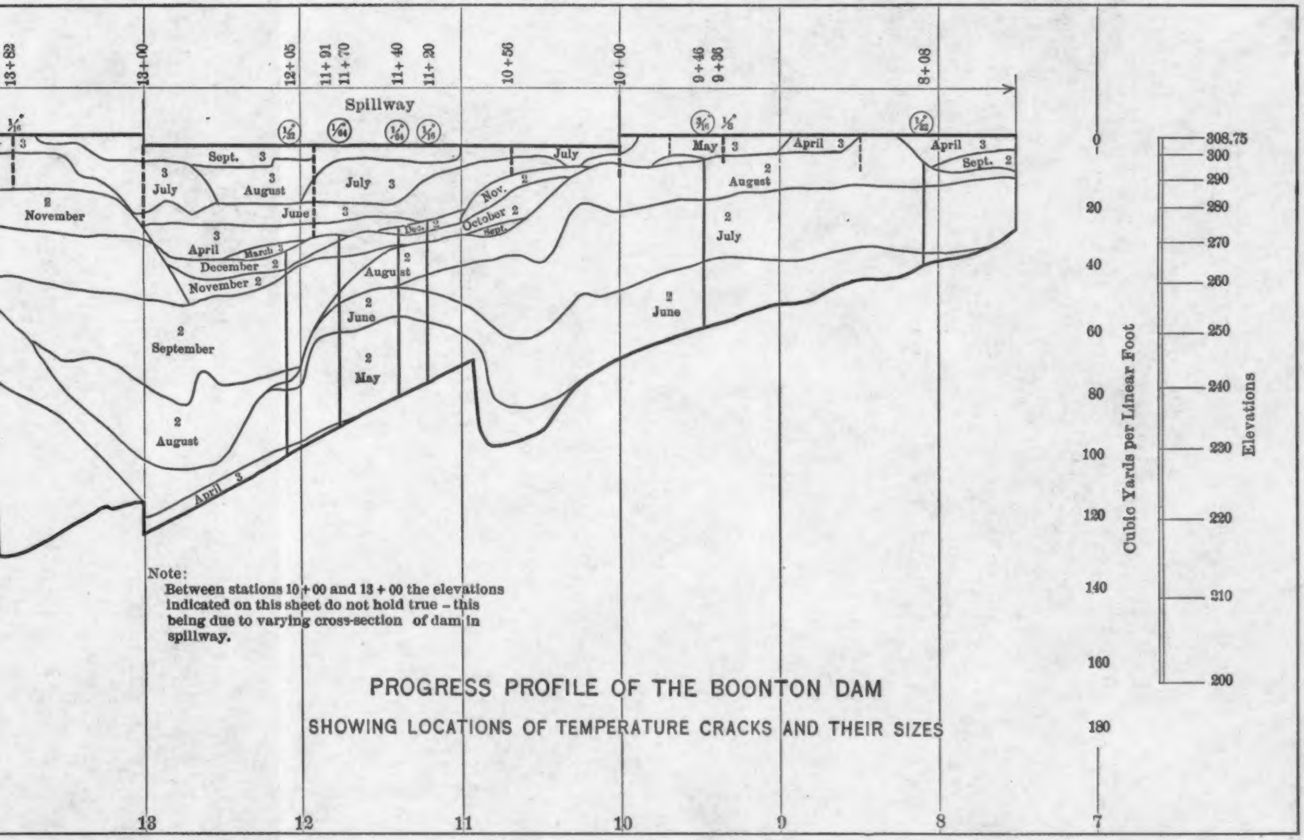


19+46
18+48
17+75
16+43
15+61
14+92
14+20
13+82
13+00
12+05
11+91
11+70
11+40
11+20
Masonry Dam 2150 Feet Long
Spillway
3 June
3 July
3 April
3 May
3 April
3 May
3 June
July 3
3 July
Sept. 3
3 August
July 3
June 3
2 October
2 November
Dec. 2
2 November
2 October
2 November
3 April
March 2
December 2
November 2
2 May
2 June
2 July
2 August
2 Sept.
2 March
2 June
2 October
June 2
2 May
2 September
2 August
2 June
2 May
1 October
2 April
2 May
2 Sept.
2 April
2 March
2 August
April 3
1 December
1 June
1 August
1 November
1 May
19
18
17
16
15
14
13
12
Note:
Between stations 10+00 and 13+00 the
indicated on this sheet do not hold true
being due to varying cross-section of
spillway.
PROG
SHOWING LO



22+72
21+97*
21+79*
21+48
21+00*
20+94
20+85
20+30
20+16
20+00
19+76
19+46
18+48
17+75
16+43
15+61
14+92
14+20
13+82
Masonry Dam 2150 Feet Long
3 May
2 August
2 July
2 June
3 March
2 August
3 April
2 September
2 August
2 March
Dec. 1
1 September
1 October
3 May
2 April
1 August
1 July
2 October
2 May
3 June
2 June
2 July
1 June
1 May
2 November
2 March
1 October
3 April
3 July
3 May
Dec. 2
2 August
2 Sept.
2 June
2 April
2 May
1 August
3 April
Dec. 2
2 November
2 October
2 Sept.
1 December
1 November
3 May
June 2
2 March
3 June
2 October
2 May
2 April
July
23
22
21
20
19
18
17
16
15
14

PLATE LIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1087.
MERRIMAN ON
EFFECT OF TEMPERATURE CHANGES ON MASONRY.



PROGRESS PROFILE OF THE BOONTON DAM

SHOWING LOCATIONS OF TEMPERATURE CRACKS AND THEIR SIZES

account of an excess of sulphur in the vulcanized rubber, or on account of the behavior of sulphur in the presence of Portland cement and its constituents, the sulphur in the insulation attacked the copper of the wires, and although perhaps not entirely destroying the wire, yet modified its resistance so that the results from the coils became valueless. Mr. Merriman.

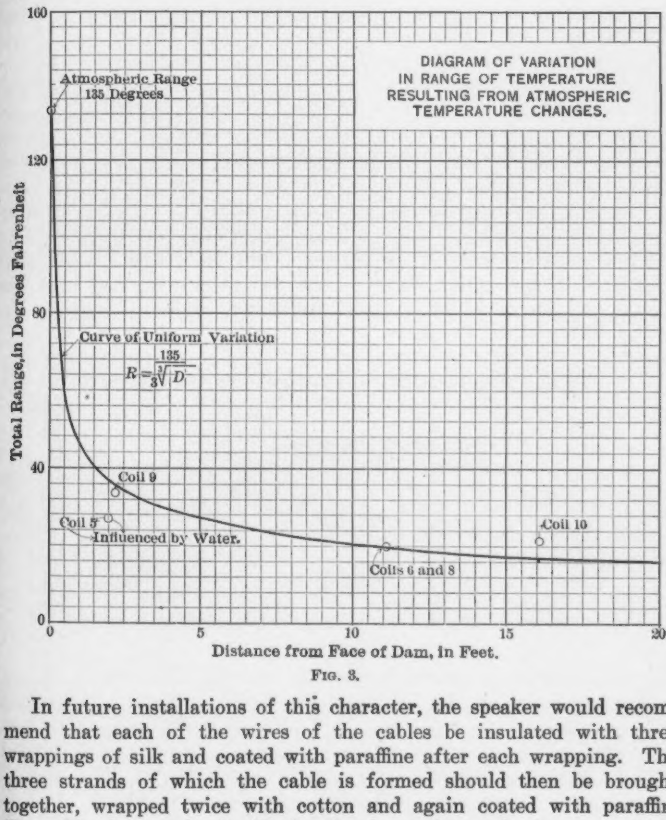


FIG. 3.

In future installations of this character, the speaker would recommend that each of the wires of the cables be insulated with three wrappings of silk and coated with paraffine after each wrapping. The three strands of which the cable is formed should then be brought together, wrapped twice with cotton and again coated with paraffin. The cable thus made should then be drawn into a lead armor, which armor should be soldered directly to the brass tube containing the thermophone coils. In this way the speaker believes that an absolutely moisture-proof lead can be obtained from the thermophone to a point outside of the masonry.

Mr. Merriman. On Plate LVI are plotted the records of the thermophone coils, together with the daily and monthly mean atmospheric temperatures. In order to avoid confusion, not all the records have been plotted, but only those showing the more continuous and uniform values. An inspection of the curves of these coils shows that all those placed prior to October, 1903, except Nos. 5 and 9, reached their minimum temperatures during the following April. These same coils reached their next minimum, however, during March, 1905, in spite of the fact that the atmospheric minimum occurred one month later than in the previous year. Coils Nos. 5 and 9, being but 2 ft. from the faces of the dam, reached their minimum temperatures not far from the time of the occurrence of the atmospheric minimum. The speaker, therefore, is led to the general conclusion that, owing to the heat generated by the cement during its setting, about one year must elapse before masonry in a dam of dimensions comparable with that at Boonton, will reach a temperature which will depend only on its position in the dam and on the changes in the atmospheric temperature. This conclusion holds true for all portions of a dam, since, if the heat generated by the setting action of the cement is being lost from any portion of the masonry, it must affect and tend to keep up the temperatures of all its parts.

TABLE 6.—RESULTS OF THERMOPHONE OBSERVATIONS, BOONTON, N. J., DAM.

Coil No.	Distance from face, in feet.	Maximum temperature, in degrees, Fahrenheit.	Date of maximum temperature.	Minimum temperature, in degrees, Fahrenheit.	Date of minimum temperature.	Total temperature range, in degrees, Fahrenheit.
5	2.0	69	Aug., 1904	39	March, 1905	30
6	11.3	64	Sept., 1904	45	" 1905	19
8	11.0	63	Sept., 1904	43	" 1905	20
9	2.5	71	Aug., 1904	34	" 1905	37
10	16.0	68	Oct., 1904	46	" 1905	22

Referring again to Plate LVI, it is easily seen that minimum and maximum temperatures near the center of the dam are reached approximately 30 days after the occurrence of the atmospheric minimum and maximum, respectively, and, moreover, the "lag" is seen to be proportional in some degree to the distance from the nearest face of the dam. Unfortunately, however, the observations do not seem to warrant the drawing of a curve of this "lag" or retardation.

Table 6 shows the principal results obtained from the thermophone observations on the Boonton Dam. It will be noted that no observations were used in the compilation of this table until the thermophones had been in place for practically a year. This was done in order to

FIG. 1.—DOWN-STREAM FACE OF BOONTON DAM, SHOWING ASHLAR FACING.

FIG. 2.—UP-STREAM FACE OF BOONTON DAM, SHOWING RUBBLE FACING AND GATE-HOUSE.

eliminate, if possible, the effect of the heat generated during setting, and heretofore referred to. The results shown in this table were then plotted on the diagram, Fig. 3, and by trial a curve most closely representing the observations was drawn. If R be the total range in temperature at any point in the mass, in degrees, Fahrenheit, and D be the distance in feet to the nearest face of the dam, then Mr. Merriman.

$$R = \frac{135}{3\sqrt[3]{D}}$$

where 135 is the total atmospheric range, as observed during the period from June, 1904, to March, 1905. During this period the minimum temperature reached was —18° fahr., and the speaker estimates that the maximum in the sun was 117° fahr., that in the shade being 98° fahr. This value, 135, is also probably not very far from the maximum atmospheric range ever attained in the latitude of New York.

Referring to the diagram, it is seen that the curve represented by this expression fits the plotted points very closely, and the speaker, therefore, has adopted it until future observations indicate that a change would be advisable. It must be noted, however, that near the minor limit the curve does not seem to fit in with the physics of the case, as it indicates that the range could never be zero, and that if the range were to be kept down to 1° fahr., the thickness of the masonry should be 182 250 ft. The speaker, therefore, prefers simply to state that the law expressed by this formula, for the present, at least, should not be considered as holding true for distances from the face of the masonry of more than 20 ft. or less than 0.5 ft.

It is well known that, after concrete and mortar have been mixed and placed in position, the temperature of the mass begins to rise. In a small cube this increase in temperature reaches a maximum about 18 hours after the cube is made, though, in a very large mass, this maximum may be delayed several days. The speaker would refer here to certain experiments made some years ago in the Lehigh University laboratory during which the temperature at the center of an 8-in. cube of 1:3 Portland cement mortar rose to 160° fahr. in 18 hours after the cube was made, the temperature of the laboratory being about 70° fahr. He would also refer to pages 414-416 of Johnson's "Materials of Construction," where it is stated that the temperature at the center of a mass of Portland cement concrete increased 93° fahr. in 7 days. The increase for the mortar cube was 90° fahr., and that for the concrete was 93° fahr., an agreement somewhat closer than one would anticipate, as the "heating" of cement would seem to depend largely on the materials of which it is composed and perhaps also on the process of its manufacture and mixing.

In the Boonton Dam nearly all the materials were deposited in comparatively thin layers, and the speaker, therefore, considers that

Mr. Merriman. their maximum temperatures were reached within 18 hours after placing. On Plate LVI is shown the record of thermophone coil No. 11 by daily readings after it was put into position. It will be noted that, starting at about 77° fahr., this coil showed a rise of about 16° to 93° fahr. in about 13 days, and that about 11° of this rise occurred within 6 days. The facts of the placing of this coil are as follows: It was set into the wall on June 24th, 1904, and was immediately covered with about 12 in. of concrete. The temperature

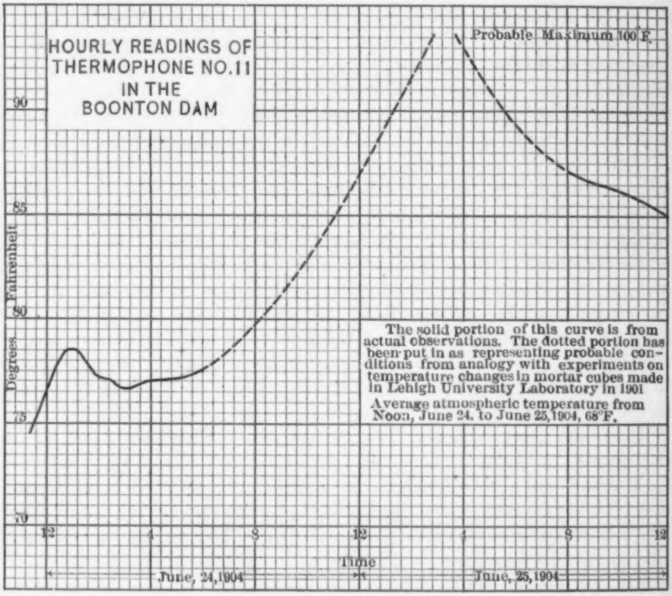


FIG. 4.

of the concrete at the time it was placed was 74.5° fahr. Seven days after placing the coil more masonry was placed above it, so that, within 14 days, it was buried to a total depth of about 11 ft. The increase in temperature recorded by this coil, therefore, was, it would seem, largely due to the heat from the masonry placed above it, and consequently it can be said that the temperature of the masonry placed above coil No. 11 and insulated from it by 12 in. of concrete already set, must have reached a temperature higher than 93° fahr.

On Fig. 4 are shown the results of hourly readings on thermophone No. 11. This coil was placed at 11.40 A. M., June 24th, 1904

FIG. 1.—GENERAL CHARACTER OF INTERIOR MASONRY OF BOONTON DAM.

FIG. 2.—COMPLETED BOONTON DAM WITH WATER PASSING OVER THE SPILLWAY.

Readings were taken until 6.15 P. M., of that day, when they were discontinued for the night. Next morning at 8 o'clock, in spite of a clear night, a minimum atmospheric temperature of 60° fahr., and notwithstanding an average atmospheric temperature of 68° fahr. from noon, June 24th, to noon, June 25th, the thermophone recorded a temperature of 87.5° fahr., a rise of nearly 10° since the previous evening. The curve of temperature for some hours after 8.00 A. M., June 25th, with a rising atmospheric temperature, showed a downward tendency, and this fact, together with the rise during the night and the general considerations of rise in masonry due to setting, as set forth in a previous paragraph, has led the speaker to indicate on Fig. 4 the probable curve of temperature for this coil between 6.00 P. M., June 24th, and 8.00 A. M., June 25th. The considerations which will explain the reasons for putting the probable maximum temperature reached at 100° fahr., are as follows: Mr. Merriman.

The Boonton Dam is constructed of cyclopean masonry, composed roughly of 50% of large stones and 50% of concrete, having proportions of 1:3:6, therefore 25% of the mass is mortar and 75% stone, whether large or small. If, now, it be assumed that the specific heat of the stone is 0.2 and that of the new mortar is 0.4, and that the temperature of the mortar alone would rise 90° fahr., it follows that the temperature of the mass will rise 36° fahr., and if the initial temperature of the mass was 74° fahr., that therefore the resultant temperature would be 100° fahr.

The average atmospheric temperature of the months during which masonry in a dam is usually placed is not far from 58° fahr., and if the chemical changes due to the setting of a cement are sufficient to induce in the masonry a rise of 36° fahr., then it is reasonable to suppose that the masonry in setting will reach a temperature of $58° + 36° = 94°$ fahr. This estimate, it will be noted, is based on the assumption that the average temperature of the sand and stone used in the construction will be the same as the average temperature of the atmosphere.

In arriving at a conclusion on this point the speaker has borne in mind the approximate character of the computations, and of the data entering them, and has preferred to state that the maximum average temperature which the masonry will attain shortly after being placed in position is 100° fahr., rather than the 94° fahr., above deduced. He has done so for the reason that it appears, from all considerations, to be true beyond a reasonable doubt that some of the masonry in a dam will reach this temperature, and that, therefore, it is conservative and on the side of safety to assert that all the masonry will do likewise. As far as observations of actual temperature cracks in a masonry dam tend to bear out this point, reference to Plate LIII will show that all the main cracks which appeared on the top of the Boonton Dam

Mr. Merriman. are found in the masonry which was placed during the warmer months of the working year. The speaker, however, considers this only as a corroborative fact, as there are many other factors which may have a great influence on the location of the cracks. Among these should be mentioned the height of the masonry, the "racks" in which it was built, and the irregularity of the foundation profile.

Before leaving the question of the records of the thermophones, it will be interesting to indicate the difference in behavior between coils Nos. 5 and 9, coil No. 5 being within 2 ft. of the water face of the dam, while coil No. 9 is within 2.5 ft. of the exposed downstream face. During the winter of 1903-1904 the water in the reservoir rose just about level with coil No. 5. Both these coils reached their maximum temperature during August, 1904, No. 5 attaining 69° fahr., and No. 9, 71° fahr. No. 5, at this time, was not submerged by more than 20 ft. of water. During the following summer, however, No. 5 reached only 54° fahr., while No. 9 reached 65° fahr., No. 5 at this time being submerged about 55 ft.

The equalizing effect of the water in the reservoir is shown by the curves on Plate LVI, which indicate that the water side of the dam is cooler in summer than the exposed face, and conversely. These curves also show that the temperatures of the two faces are equal twice each year, in December and April.

The setting of cement is the physical phenomenon resulting from chemical changes within its mass. By those who have made a study of the subject, these changes have been likened to the arrangement of the molecules into a definite and fixed form, resembling crystallization to some extent. It has been seen that the process of setting is accompanied by the evolution of heat, and Johnson* says:

> "In fact, with quick-setting cements the temperature curve is a truer index of the setting period than the mechanical tests of firmness which are usually employed for this purpose."

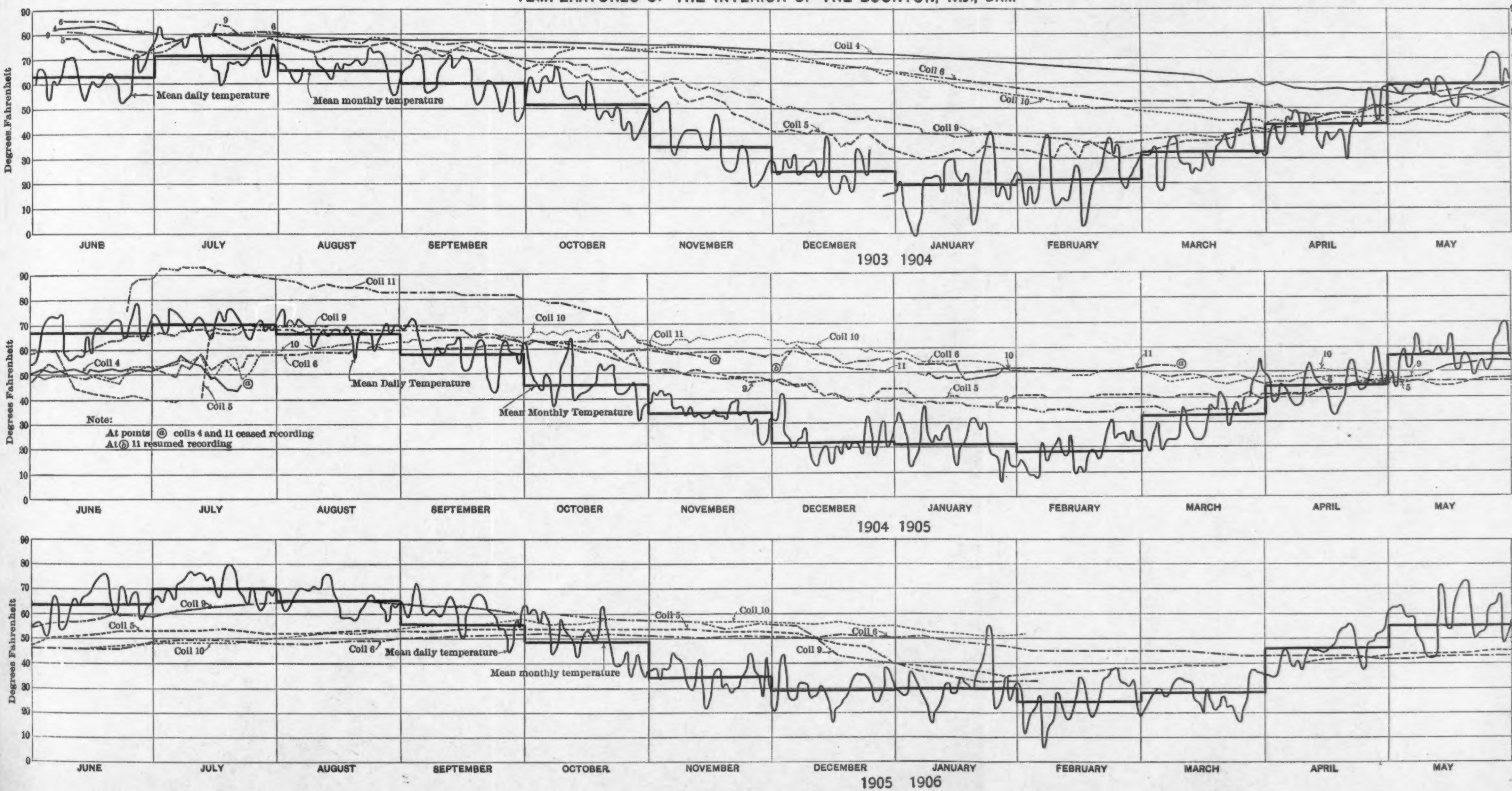
Again, on page 416, he says:

> "With slow-setting cements the rise of temperature cannot be observed with accuracy, and is smaller in amount than in case of quick-setting cements. In such cases the heat developed is dissipated because of its slow generation, and does, therefore, not become sensible to thermometric measurement."

From all points of view, therefore, it appears to be probable that Portland cement in setting assumes its final definite form and shape at a time when its setting temperature is a maximum. The speaker believes it to be an undisputed fact that the range of temperature which produces the maximum stress in a body, provided only that the

* "The Materials of Construction," New York, 1906, p. 415.

TEMPERATURES OF THE INTERIOR OF THE BOONTON, N.J., DAM



body is not free to contract or expand, is the difference between the highest and lowest temperatures to which the body will be subjected. In view of all the evidence, therefore, he submits that in a masonry dam the maximum range in temperature which causes tensile stresses in the masonry is the difference between the temperature at which the cement assumes its set and the lowest temperature which may subsequently be reached. From this it at once follows, if the average setting temperature be 100° fahr., that the minimum total tensile stress producing range in any portion of any dam cannot be less than the difference between 100° fahr. and the yearly mean temperature of the atmosphere at the place where the dam is built. In the latitude of New York this minimum range, therefore, is 52° fahr. Mr. Merriman.

Fig. 5 is a cross-section of the Boonton Dam on which have been plotted the curves of probable total temperature range within its mass, no attempt having been made to allow for the influence of the water in the reservoir, as the information at hand is too meager to warrant such a study. The problem, therefore, has been studied from the point of view of a simple masonry wall of the cross-section shown.

The atmospheric range of 135° fahr. is undoubtedly true on the exterior faces, but 2 ft. within the dam it is found that the application of the law of variation of the range in connection with the setting temperature of the masonry gives a range of only 72° fahr. It is also found that it is necessary to go 18 ft. farther into the dam to find a point where the range is 60° fahr. Fig. 5 is interesting in that it exhibits a condition of affairs in the center of a masonry dam almost diametrically opposed to that which has heretofore been believed to be true.

In connection with the measurements of the cracks in the Boonton Dam and the temperature ranges herein indicated, the speaker endeavored to obtain some idea of the tensile stress which the masonry actually carries. In order to do this it became necessary to make certain assumptions, as follows:

a.—The top width of a temperature crack is the same as its width at a point 2 ft. toward the interior of the dam. Were this not the case one might expect very shortly to see every concrete dam begin to "peel off." The crack is prevented from assuming the size indicated by the range in temperature at the surface by shearing forces which stress the masonry in directions parallel with the axis of the dam.

b.—That the coefficient of elasticity of the masonry is 2 500 000 lb. per sq. in.

c.—That the range in temperature which has produced the cracking and stresses is 72° fahr.

d.—That the coefficient of expansion of the masonry is 0.0000055 per degree, Fahrenheit.

Mr. Merriman. The total visible cracking has been seen to aggregate 3.50 in., and the length of the masonry is 2 150 ft.

With these data it appears that, had no cracking occurred, the tensile stress in the masonry would have been 990 lb. per sq. in.; had the masonry been free to move, the contraction would have amounted to 10.20 in., and that since an elongation of 6.70 in. remains unaccounted for, the tensile stress in the masonry must now be about 650 lb. per sq. in., a result which agrees fairly well with what might be supposed to be the tensile strength of a large mass of masonry if the strength of small test pieces of its mortar at one year showed by test 380 lb. per sq. in.

If tensile stresses exist in a masonry dam and act on planes at right angles to its axis, there is no admissible reason for believing that such stresses do not exist and act with equal intensity on all planes and at every angle. This being the case, the stresses due to this cause should not be overlooked when the design of a masonry dam is under consideration, particularly when it is remembered that the forces which act on a dam cause tensile stresses within its mass.

An entirely independent confirmation of these results has come to hand in the interesting facts presented by Mr. Honness, who has pointed out that the cracking in the Cross River Dam probably did not extend entirely through its cross-section because the leakage, when the water stood below a point 46 ft. below the dam crest, was very slight. It is probable, therefore, that the cracks did not extend much more than 46 ft. below the dam crest, and it would seem to follow that at this point the temperature range was just sufficient to overcome the tensile strength of the masonry. Referring to Fig. 5 it will be seen that the temperature range at a point in the center of the dam and 46 ft. below its crest would be not far from 61° fahr., the cross-sections of the two structures being very similar. This range in temperature, assuming the same constants as previously used, would indicate that the masonry had a tensile resistance of about 800 lb. per sq. in., which agrees very well with that determined for the Boonton Dam when all the differences are considered, and when it is remembered that this dam was not a year old when the cracks were observed. Its interior temperature, therefore, was higher than it will be later, and consequently the actual range in temperature was less than that used in the computations. Such a condition would make the apparent strength of the masonry somewhat higher than its true value. The steel rods used in the upper portion of this dam may also, possibly, have had some influence on the size and positions of the cracks, though the speaker does not feel that the introduction of steel into a masonry mass can have any other action than to prevent the localization of cracking. The total cracking due to temperature changes must be the same, whether or not steel is used.

Mr. Merriman.

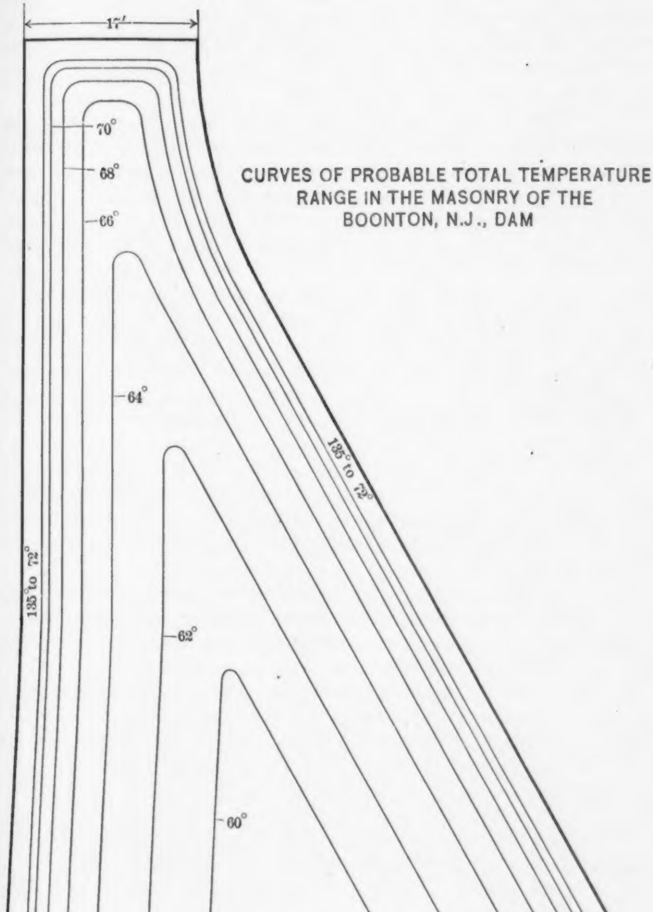


FIG. 5.

Mr. Merriman. The theoretical deductions thus made on a large masonry mass seem to be fairly well confirmed by the physical facts of its behavior after visible surface cracking has occurred. Yet it is by no means certain that they are more than appproximately correct, not only on account of the uncertain nature of the data, but also on account of the many factors entering the problem for which no allowance in the present state of knowledge can be made.

In addition to the thermophone observations at Boonton, the attempt was made to ascertain whether or not there was any change in length between the ends of the masonry structure due to seasonal temperature changes. For this purpose lead plugs were inserted near each end of the masonry and about 2 000 ft. apart. At right angles with the axis of the dam and on a line with each of these plugs two heavy posts were set into the ground, penetrating well below the frost line. Using a transit, a point was then put on each of them in line with a point on the lead plug. Frequent observations with the transit failed to detect any shortening movement; such motion as was detected may have been due to errors of observation, and the information gained was negative only.

In concluding his discussion of the Boonton results, the speaker would summarize as follows:

1.—About 12 months must elapse after the placing of masonry in the cross-section of a dam comparable with that at Boonton before the heat generated by the chemical changes incident to the setting of the cement becomes lost to an extent sufficient to render its existence a negligible quantity.

2.—After the expiration of 12 months, minimum and maximum temperatures near the center of the masonry are reached approximately 30 days after the occurrence, respectively, of the seasonal atmospheric minimum and maximum.

3.—The range in temperature at any point in the interior of a masonry dam, due to external temperature changes, is dependent on the exterior atmospheric temperature range, and on the distance of the point from the face of the dam. For distances up to 20 ft. and perhaps for greater ones, the interior range varies inversely nearly as the cube root of the distance from the face.

4.—Immediately after concrete and mortar are placed in a dam, the temperature of their mass rises and probably reaches a maximum in about 18 hours after the placing. This maximum is about 100° fahr. for cyclopean masonry.

5.—It may be inferred that the constituents of the mortar and concrete assume their final form and shape at the time when their maximum temperature is reached. This being the case, it would appear that nearly all portions of a masonry dam are under a tensile stress, practically from the time the materials composing it are placed in their position, and consequently it seems to follow that the stress producing

range in temperature is more nearly equal for all portions of the structure except those very close to the exterior of the mass than has hitherto been supposed. This deduction follows from the consideration that, if the maximum temperature attained is 100° fahr., then the stress producing range is the difference between 100° fahr. and the lowest temperature subsequently attained. Mr. Merriman.

6.—Approximate computations indicate that the ultimate tensile strength of the mass of a masonry dam constructed of Portland cement is not far from 700 lb. per sq. in.

7.—From the evidence at hand, it would seem that the use of a slow-setting cement would tend to keep down the setting temperature, and, therefore, it might be expected that a structure made of such a cement would show less cracking than one made of a cement which takes its set more rapidly.

In Table 4 the author gives the coefficient of expansion for masonry in large masses. This coefficient of expansion, 0.00000307, is somewhat smaller than that which has been used heretofore in the design of masonry structures, in fact, about one-half, and the speaker would suggest that this difference may be accounted for, at least, in part, by the fact that, in deducing his value, the author used the range in atmospheric temperature, whereas the temperature range which was effective in the mass of masonry was materially less.

The present-day design of massive masonry structures is deficient in so far as a proper knowledge is had of, and proper allowance is made for, the effect of the tensile stresses due to temperature changes. These stresses, when taken together with the other tensile stresses resulting from the forces acting on the mass, may give rise to resultant stresses in excess of conservative practice; therefore, all facts bearing on this question are of value, and the author is to be congratulated for having presented in such concise form the results of his extended observations on the highest masonry dam in the world.

William Lowe Brown, M. Am. Soc. C. E.—As a short account of the cracks which appeared in the Assouan Dam may be of interest, the speaker will describe them to the best of his recollection. Mr. Brown.

Mr. Gowen's paper gives a very clear account of the cracks which appeared in the New Croton Dam, and of the very careful measurements which the author had taken for so long a period. Such painstaking work is always of great value to others, and engineers should be grateful to him for what he has done.

Mr. Gowen concludes by suggesting that, in order to avoid cracks, certain precautions should be taken; these may be summarized as follows:

(*a*) Sudden changes of section should be avoided.

(*b*) The work should be carried out in horizontal layers, and all racking and stepping avoided.

(*c*) The work should not be carried on in hot weather.

Mr. Brown. In building the Assouan Dam, owing to unavoidable conditions, these rules were violated in a very flagrant manner, especially in the lower half of the dam, as will be seen from the following:

(*a*) It was impossible to avoid sudden changes of section, for provision had to be made for allowing the River Nile when in flood to pass without damaging the structure, and this could only be done conveniently by leaving sluices through the body of the dam. There were 180 of these sluices, 120 of which were 23 ft. high and 6½ ft. wide, and the remainder 12 ft. high and 6½ ft. wide. In addition to these, there were buttresses between each set of ten. The greatest variation in section, however, was caused by the very irregular nature of the foundation line, which was cut up by five distinct river channels, divided by high points which were only flooded at high Nile; and the irregularity was increased still further by the fact that the rock at the bottom of the various channels was usually much softer than that between them. Consequently, in order to get a solid rock foundation, the excavation in the channels had to be carried to a greater depth below the original rock line than in the higher portions; so great, in fact, was this sudden drop at one point, that it was deemed advisable to cut away the good rock at the sides in a series of steps in order to make it less sudden.

(*b*) It was impossible to carry up the lower portion of the work in horizontal layers, for only two of the river channels could be blocked in any one year, and it was necessary to complete the excavation and carry up the masonry above the mean river level in these channels in one working season; the result was that, in the earlier stages, each season's work consisted of a large mass of masonry in the newly exposed foundation and a straggling piece of work above that of the previous year, and the first section was almost completed before the last part of the foundation was commenced.

(*c*) It was impossible to avoid carrying on the work in the very hottest weather, for the working season was only about eight months—December to August—and as, during the first four of these months, most of the time was spent in excavating the foundation, it was necessary to do the bulk of the work, in the lower portions of the dam, in May, June, and July, during which months the mean temperature remained above 90° fahr. The maximum midday shade temperature was very frequently 110° fahr., and occasionally 122° fahr.; in the sun it was very much hotter (170° fahr., or more). The minimum winter temperature is usually about 50° fahr. and occasionally it goes down to 40° fahr.

Being built under such conditions, it would be reasonable to suppose that the Assouan Dam would be very badly cracked. This, however, was not the case. There were perhaps six or seven cracks, altogether, but none of them was serious. The largest crack occurred at one of the most sudden changes of section, due to a high step in the

foundation, at the side of a sluice and in close proximity to one of the Mr. Brown. buttresses. This crack extended from the top of the dam almost down to the rock foundation, and was about ¼ in. wide at the foundation and rather more at the top, in cold weather. It extended through the dam from side to side.

Calculations show that in a dam of this length—6 500 ft.—the total amount of contraction would be about 1 ft., assuming the range of temperature of the masonry to be only 30° fahr. In point of fact, the aggregate width of all the cracks could not have been more than from 1 to 2 in. The masonry, however, must have contracted by an amount commensurable with that given by the calculation for the stress set up by the tendency to shrink. Had there been no cracks, this would have been 375 lb. per sq. in., if the modulus of elasticity were 2 500 000 lb. per sq. in., and if it were higher, which is much more likely, the stress would be greater. Even if the masonry remained under a tension of 200 lb. per sq. in., the aggregate width of the cracks must have amounted to about 6 in.; it is probable, therefore, that there were a great number of microscopic cracks which were not visible.

It has frequently been observed that in a concrete structure there are more cracks than in a similar structure of stone masonry. This is probably because concrete can only relieve itself of stress by a fracture of the whole mass, while a masonry structure can give way by creeping and shearing between the component stones and their beds.

The amount of leakage through these cracks was very small, and, at the worst one, only amounted to about 5 or 10 gal. per hour, and decreased very considerably after the reservoir had been full for some time. The small leakage compared with the size of the cracks is usually assumed to be due to the irregularity of the fracture, and to the fact that the width is much less in the heart of the masonry, where the temperature changes are much smaller; this is probably the correct explanation.

A reason for the decrease in the leakage, after the reservoir has been full for some time, has suggested itself to the speaker; it is that the masonry on the up-stream face swells when it becomes wet, and in so doing closes up the crack.

That swelling is a very real factor was clearly illustrated in the observations, referred to by Mr. Gowen, which were made by Sir Alexander Binnie on a concrete beam 100 ft. long and 1 ft. square in section. The changes of length due to changes in the humidity of the concrete frequently almost entirely masked those due to changes of temperature, and it was only after arrangements had been made for measuring the amount of moisture in the concrete, every time an observation of the length of the beam was made, that a standard of comparison could be established for the various observations of length, and that the changes of length due to changes of temperature could be distinguished from those due to changes of humidity.

Mr. Brown. With regard to Mr. Gowen's paper, there are two points which might be mentioned. The addition of a foundation line to the longitudinal view of the dam would tend to show more completely the changes of section from point to point, and might explain the position of the cracks.

The probable reason why the coefficient of expansion of the masonry as found by Mr. Gowen is so small has been pointed out by Mr. Merriman. It is certain that the range of temperature of the masonry itself must have been less than that of the atmosphere, and, had it been possible to observe the former instead of the latter, a very accurate value for the coefficient might have been obtained. That this would have been the same as that shown in Table 4 is not very probable, for there is no reason why various kinds of concrete and masonry differing so greatly in their component materials should show any invariability in their coefficients of expansion.

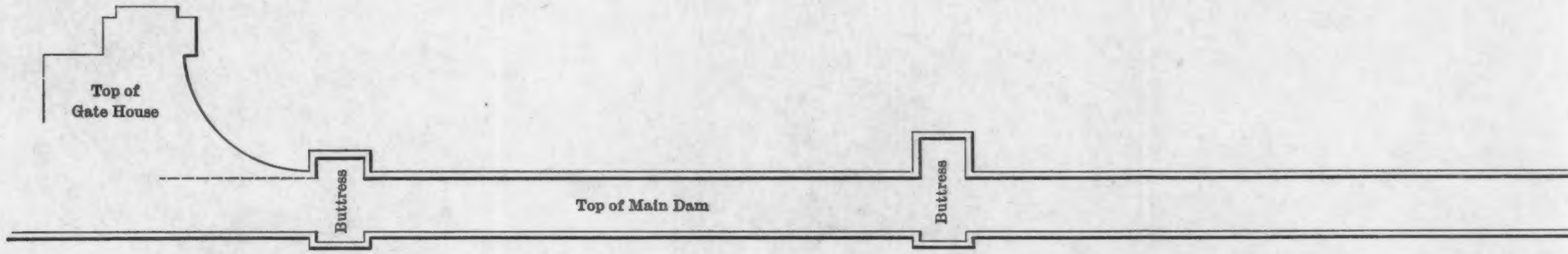
There is one sentence in Mr. Gowen's paper to which the speaker wishes to take exception. After stating that the coefficients he obtained may have been affected by the gradual drying out of the masonry, he continues: "Possibly the higher coefficients (the first two in Table 4) may be due in a measure to this." The first value of the coefficient is obtained from the increase of length due to a rising temperature, and therefore any shrinkage due to drying would have tended to reduce the coefficient and not to increase it.

Mr. Merriman has stated that, in the use of the thermophones on the Boonton Dam, there was great difficulty with the insulation of the wires. He suggests that the wires be incased in lead. This was done in the case of the resistance thermometer built into the Assouan Dam, and there was no difficulty in getting accurate temperatures.

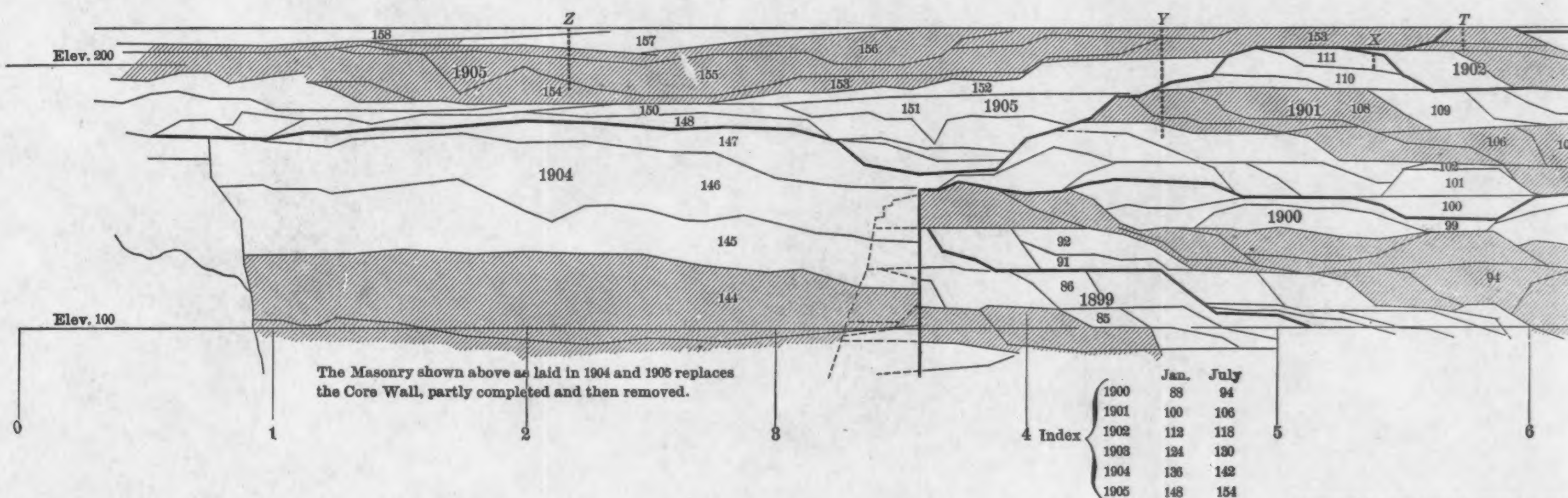
Mr. Gowen. Charles S. Gowen, M. Am. Soc. C. E. (by letter).—The writer feels that the Society is to be congratulated upon the comprehensive and elaborate discussions that his paper has occasioned. He takes this opportunity to thank Messrs. Honness and Merriman especially, for the information afforded in their descriptions of temperature effects at the Cross River and Boonton Dams, and for the specially interesting and instructive results which the thermophone observations at Boonton have afforded, as well as for the elaborate deductions concerning the action of setting mortar in connection with varying air temperatures and their effect on the structure in question.

Their general conclusions would seem to corroborate those of the writer as expressed in the paper, while the thermophone records afford very complete information, which the writer deplored as lacking in his remarks, as to the results of temperature changes other than those due to the air.

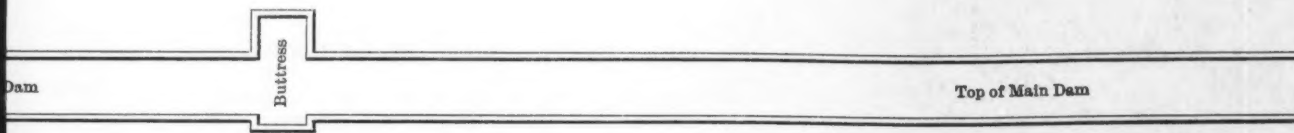
As such complete information has been supplied by Messrs. Honness and Merriman regarding the cracks in the Cross River and Boonton



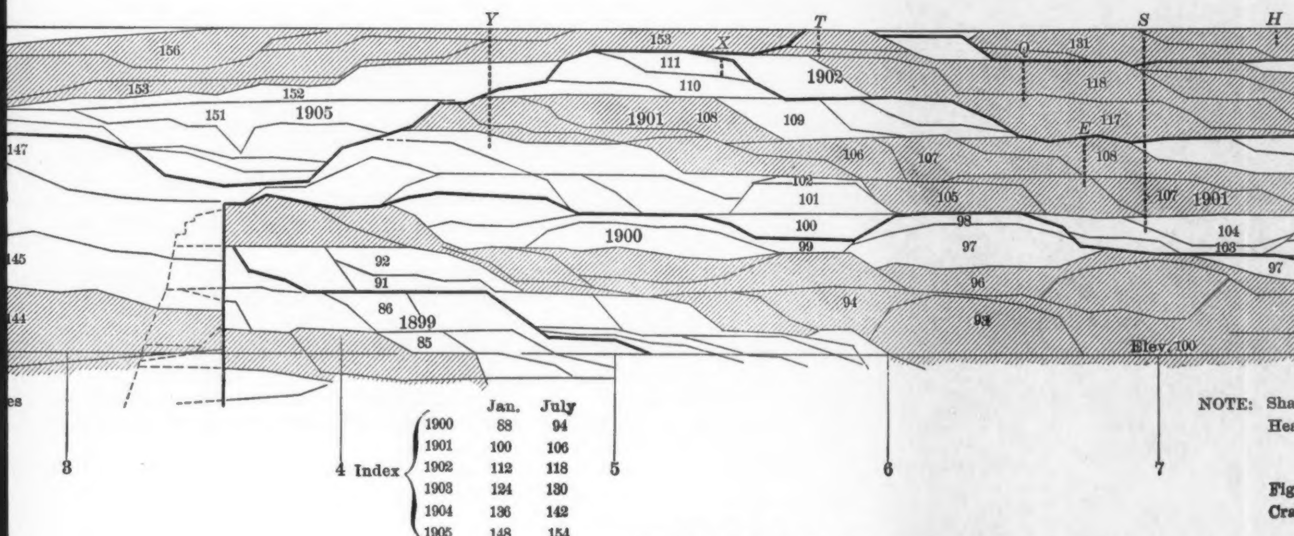
PLAN OF MAIN DAM



PROFILE OF MAIN DAM LOOK



PLAN OF MAIN DAM



PROFILE OF MAIN DAM LOOKING DOWN STREAM

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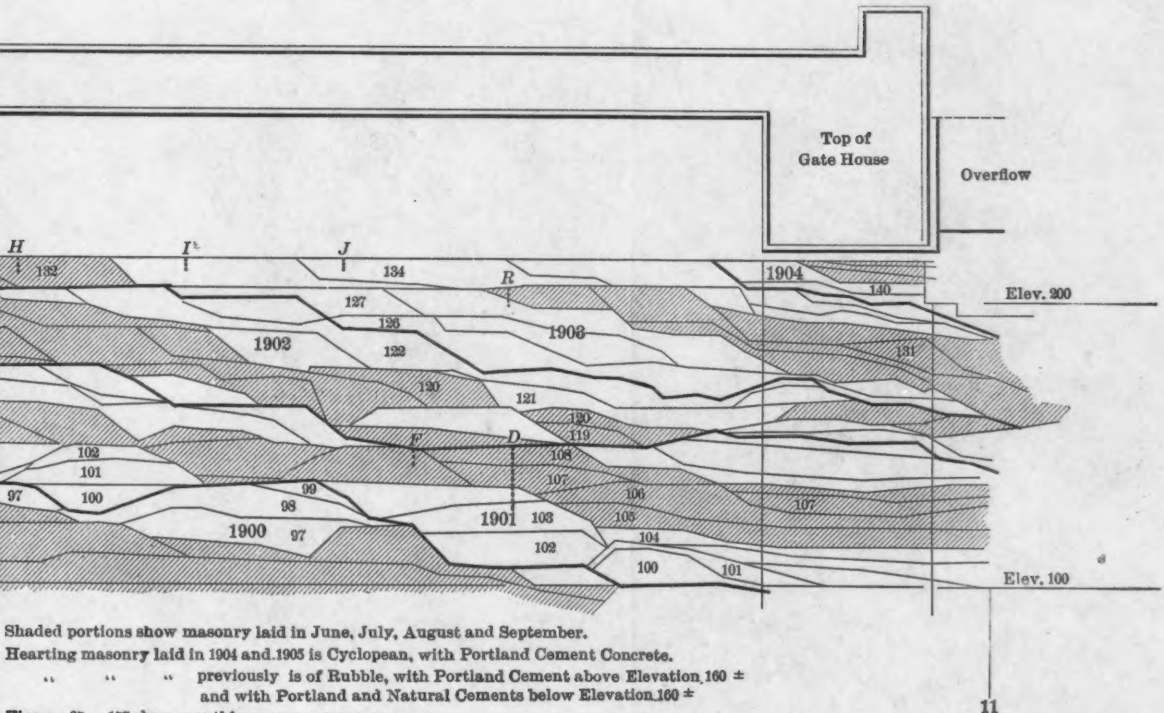
PLATE LVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXI, No. 1087.
GOWEN ON
EFFECT OF TEMPERATURE CHANGES ON MASONRY.



Shaded portions show masonry laid in June, July, August and September.
Hearting masonry laid in 1904 and 1905 is Cyclopean, with Portland Cement Concrete.
" " " previously is of Rubble, with Portland Cement above Elevation 160 ±
and with Portland and Natural Cements below Elevation 160 ±
Figures 85 — 157 show monthly progress masses.
Cracks are indicated by letters thus *X. Y.* and broken lines.

Dams, it is perhaps well to supplement the original paper by furnishing similar information concerning the New Croton Dam. **Mr. Gowen.**

The cracks referred to and described in the paper are those on which observations were made, and they had developed either in the winter of 1901-02 or previously. The succeeding winters, however, produced additional cracks in the masonry laid subsequent to that in which the studied cracks occurred, and the profile, Plate LVII, includes the main dam section only. The overflow section and profile and the significant cracks which developed in it are shown in the original paper, and such cracks as may have developed later are not produced here as they are of little or no consequence.

The cracks noted on this profile include all those in the main dam to which any importance was attached. If others occurred they were of such character that no record of them was made, and the writer does not recall any such.

The shaded parts of the profile show the masonry laid in the warm months, June, July, August, and September, and attention is called to cracks *S, Q, T, Y,* and *Z,* which developed in the winters of 1902-03, 1903-04, and 1904-05, as shown. Attention is also called to the positions of *S, Q, E,* and *D,* relatively to the warm-weather masonry. *Y* and *Z* also show significantly, in the same connection, as in the case of *Z,* the layer of cold-weather masonry is not heavy enough to protect the masonry below materially from the influence of temperature change, while the section of the dam at this point is small, in comparison with the buttress section on either side.

Crack *S* developed late in the fall of 1903. During the following winter it extended lower into the masonry laid in the preceding two years. It was caulked with grout and lead in the spring of 1905, before the water had risen permanently behind it, and since then has shown in the winter some spraying seepage at certain elevations on the down-stream side. In the warmer months no leakage shows. In damp weather sweating discolors the down-stream face of the dam at various points, and some of it may come from this and other cracks. When the sun reaches the face of the dam, however, the dampness disappears and at no time during the warm months is this sweating enough to create any flow of water, or even a marked dampness.

Crack *S* is of interest in that it extends down from the top of the dam about 70 ft., and in that its development was continued through two if not three winter seasons, indicating clearly that its later developments were due very largely and directly to atmospheric changes in temperature, as the changing temperature of the masonry due to the aging of the cement mortar could not have been great, although chemical action in Portland cement mortar, in the writer's opinion, is by no means limited to one or two years after its use. However, such action is so moderate and gradual that it may have no

Mr. Gowen. influence, practically, on the mass temperature after the first year, as seems to be indicated by the thermophone. The parts of the dam in which the above-noted cracks, *S, Q, T, Y,* and *Z,* occur were built of Portland cement which showed quick-setting and high-breaking records, as doubtless did the cements used at Cross River and Boonton, and, as Mr. Merriman remarks in his general conclusions, it would seem that, with slow-setting cements, setting temperatures would be lessened. Mr. Honness touches the same point when he alludes to the temperature changes as due to greater rapidity of construction causing greater retention of heat and greater temperature range in setting of masonry.

It would seem to the writer, therefore, that a comparison of this New Croton Dam profile with the Boonton and Cross River profiles, shows similar results arising from similar conditions. The Cross River profile shows 4 cracks. They developed in the winter, in masonry laid in the previous summer. The three main cracks, it would seem to the writer, occur at intervals notably regular, and all the cracks tend to locate or occur at the junction of racked masonry masses, though this tendency is not equally marked in every case.

At Boonton the cracks also developed with cold weather, and in warm-weather work. The effect of reasonably heavy masses laid later in the season in protecting summer work is remarkable, as is the tendency of cracks developed in the masonry of one season to extend down into the warm-weather masonry of the previous season. This is also shown in the case of Crack *S* at the New Croton Dam. The uniformity of the crack intervals, shown on the profile, in the masonry laid in the third season (summer), as noted by Mr. Merriman, is, under the conditions, what should be expected. The racks occasioned by the work done in August, 1903, which work remained in that shape during the winter following, have cracks at their feet and a crack shows also at the foot of the September rack of the same season; and it would seem that, among other causes that influence the formation and location of cracks, the varying section must be considered, and that in fact the question of crack development may be summarized as due chiefly to the following specific causes, viz.,

Masonry laid in warm weather;

Masonry laid with mortar in which quick-setting, finely-ground, high-testing cement is used;

Irregularity in sectional area, whether due to the plan of the structure or to the way in which the building progresses.

The following are special descriptions of Cracks *S* and *Y*, of the New Croton Dam profile:

S.—This crack developed in November, 1903. It was a vertical crack extending down to the invert of the gallery, Elevation 207±, cracking the facing stones in the up- and down-stream cornices, and passing freely the water forced in to test it. In December, 1904,

the crack had increased in size, and widened, and the cracked stones in the up-stream face showed plainer and the facing stones were found to be cracked in the 28th course below the three dimension-stone courses of the coping. In November, 1905, this crack had extended down to the 34th course of facing stone below the dimension-stone courses. Cracked stones showed in the 24th and 36th courses on the up-stream side and in courses on the down-stream side, where, however, the crack is more limited in extent, and extends downward only about 50 ft. from the top of the dam. In its upper reaches, this crack showed in cold weather on the top of the dam and on the up-stream side a width of at least $\frac{1}{8}$ in. and its development on both faces of the dam indicated very clearly from the number of cracked stones in its line the tremendous force acting to produce this disruption. Various tests, from time to time, begun shortly after its first development, showed that the crack would pass water freely when poured in from the top, the water finding its way out on the faces of the dam. In March, 1905, the water in the basin rose to Elevation 174, or to an elevation of about 25 ft. above the bottom of the crack on the up-stream side. This elevation of the water was maintained for some weeks, and was then gradually lowered. On September 1st, the water which had been kept at about Elevation 180 for much of the summer was drawn off. Mr. Gowen.

Early in this season some seepage was noticeable on the down-stream side at the lower end of the crack. While this was particularly noticeable in March, it became less as the weather grew warmer, and on bright days disappeared.

Advantage was taken by reason of the access afforded by the water in the basin to examine and treat this crack. It was grouted early in April from Elevation 160± to the top of the dam. Below Elevation 160 the crack was too fine to grout. This grouting was accomplished by plastering the crack with clay for successive heights of about 2 ft. beginning at the bottom. Retaining dams were thus formed above which this grout of neat cement was poured. In this way, 20 bags or 5 bbl. of cement grout were introduced into the crack. This was considered quite a satisfactory operation, although none of the grout showed on the down-stream side.

Later in the season (June), when the water had lowered and access to the lower reaches of the crack could be had, further attempts to introduce grout in this part were made, but without success. The whole length of the crack was then channeled by stone cutting the face of the masonry, a cut 1 to 2 in. in width and 2 in. in depth being made. This crack was caulked with lead and was then left to be recaulked before the basin refilled in case further indications of the widening of the crack occurred.

Y.—This crack developed late in the autumn of 1905, shortly after the completion of the upper stretches of the masonry in which it

Mr. Gowen. occurs. It extends from the new masonry down into masonry which is more than four years older, and which, up to this time, had withstood temperature changes without cracking. At the time of writing the foregoing description (February, 1906), the crack was 45 ft. deep on the up-stream side, was on the top easily ⅛ in. wide, and took grout freely. It has been caulked with lead on the up-stream side as was Crack *S*. The facing stones in both the dam faces show the customary vertical cracks.

Mr. Merriman reaches the conclusion that, in a structure comparable to the Boonton Dam, and built, it is assumed, of similar materials, it will take about a year for the masonry to reach a temperature which will depend only on its position in the dam and the atmospheric temperature changes, and he refers to certain experiments on the temperature of setting mortar. The writer would also call attention to some experiments made at the Watertown Arsenal on 12-in. cement cubes. Twenty-nine different cubes were tested. These represented 27 different brands of Portland cement, and included 4 tests of "old" cements, meaning, it is presumed, cements which had been ground and in stock for an extended time. The average initial temperature of the 29 tests was 25° cent. = 77° fahr., and the maximum temperature (average) developed, reached 85° cent. = 185° fahr., giving an average rise of 108° fahr. in an average time of 13 hours.

In the case of the 4 samples of "old" cement, the average initial temperature was 16° cent. = 59° fahr.; the average maximum temperature was 70° cent. = 158° fahr., and the average rise was 99° in an average time of 19 hours.

The above results, as well as those on which Mr. Merriman bases his calculations, are derived from finely ground, high-burned, quick-setting cements, cements which it is assumed are all or mostly the product of the modern rotary-kiln process. Mr. Brown, in his interesting references to the Assouan Dam, states that comparatively few cracks developed, notwithstanding great irregularity in the construction progress, as well as a somewhat large range in temperature extremes. It is suggested that, among other causes which may have tended to such a result, the cement used was slow-setting and well-seasoned, and therefore of comparatively limited or small chemical intensity in its action. Mr. Deacon, in his accounts of the Vyrnwy Dam, states that great pains were taken to keep the masonry at a mean level as it progressed. Progress was comparatively slow and moderate, and the cement used was seasoned after delivery at the dam by exposure to the air in beds about 6 in. thick, which were moved or turned daily for two weeks before use. No cracks in the structure resulted. The range in atmospheric temperature in Wales, however, is hardly as great as in New York or Egypt.

As to Mr. Merriman's sections showing curves of temperature range in the Boonton Dam, it may be said that the actions of the cracks at the New Croton Dam suggested conditions which would have

Mr. Gowen.

been explained under the varying temperature range theory, and among these conditions it was assumed, at the time the investigations were in progress, that the cracks were of less dimensions in the interior than nearer the outside of the masonry.

As to the coefficient of expansion, 0.00000307, derived by the writer, Mr. Merriman may be right in his contention that the effective temperature range is less than that shown by the atmospheric changes, but the masonry section in which the observations were made consisted of heavy dimension stones in a comparatively small cross-section, and, as noted in the paper, the total expansion must have been affected by the drying out or shrinkage cracks in the cross-joints between the stones which had developed before the observations were begun. The temperature range in the section in question as assumed from the observed atmospheric temperatures was at the maximum, 72°, as shown in the table. It would not seem that Mr. Merriman's contention, that the actual range was materially less, holds, as his deduced curves of total temperature range in the Boonton Dam where the section is at least 50 ft. thick indicate a minimum of but 60 degrees. The writer's coefficient, of course, was offered for what it was worth, and as corroborating approximately, in a practical way, the results of scientific experiment, but the result, it would seem, also indicates and confirms this point, that masonry masses do not show the same rates of change as are indicated from expansion experiments made on the usual small scale. If it were not so, we should expect to find a total amount of contraction approaching the total theoretical width of cracks in such structures as the dams under discussion, and with no resulting internal strains.

Mr. Brown, in his summary of the author's conclusions, seems to be in error in that he states that masonry work should not be carried on in hot weather. The author's limitations as to warm-weather work were confined to uniform sectional progress and to covering such work with heavy layers laid in the colder months.

Mr. Brown alludes to the lack of the foundation line in the longitudinal profile of the New Croton Dam, given in the paper, and suggests, that, were this line given, the changes of section might be shown more completely, and possibly further explanation of the position of these cracks would be afforded. It is not thought, however, that such would be the case, as the cracks do not anywhere extend below a point when the section is more than 40 ft. thick while the section at the foundations in the main dam varies from 220 to 130 ft. in thickness, according to the depth to which the foundation work is carried. Moreover, this foundation masonry was built very largely of natural cement mortar and was laid several years before the masonry which developed cracks. Its influence, therefore, on the formation and position of these cracks, could not have been significant. At the point where the Crack *S* occurs, the height of the masonry section above its base is about 270 ft.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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TRANSACTIONS

Paper No. 1088

THE MINING OF METALS.

An Informal Discussion at the Annual Convention, June 25th, 1908.

By Edwin N. Hawkins, Esq.

Mr. Hawkins. Edwin N. Hawkins, Esq.*—Colorado mining methods and procedure, and the efficiency of its miners, are everywhere synonyms for excellence and intelligence in the field of mining. This is the result of long years of schooling in battling with the problems the industry has presented in this State. No other commonwealth has encountered an equal number of problems for the profitable treatment of refractory ores. Starting with the first lode mining in Gilpin County, the pioneers speedily came to the sulphides, which baffled their attempts to win the gold and silver by the only method then known, amalgamation with mercury. It required several years for even small progress toward rendering the metals partially free by roasting the sulphides containing them. The building of the smelting works at Black Hawk, in Gilpin County, thus marked an era of advancement. Following closely, the Leadville District was opened, together with several minor camps, and at once it became necessary to apply smelting methods to recover the values. Placer mining had preceded both in this and the Gilpin section, having had its birth with the first discovery of gold in the State, on the banks of Cherry Creek, within the City of Denver.

Even with the application of shaft-furnace smelting in Leadville, however, the problems of the necessary recovery, to render mining profitable, were only beginning to be met. Trained metallurgists, schooled in Germany, were the initial brains and energy, and the leaders in the infant industry. They were soon joined by young men

* Member, American Institute of Mining Engineers.

from technical schools, and the improvements made in the recovery of values were rapid. Thus progress in metallurgy has been intimately and constantly connected with the growth of the mining industry from its very beginning in Colorado. Mr. Hawkins.

The ore products, first valued for their gold and silver contents only and paying the miner less than what would now be an enormous treatment charge, were of necessity high in grade. Gradually the contained lead and copper were paid for, and added to the gross value of the ton, and finally the miner began to share also in the value of the more desirable fluxing constituents of his ore product. That which in 1879 was a small weight in tons and high in grade was added to in the growth of ten years by many thousands of tons monthly, which brought the producer upwards of $10 more per ton in first or gross valuation, by reason of these new features and an accompanying reduction in the treatment charge as well. The progressive smelting establishments grew in tonnage, importance, and profit; and the development of some of the newer districts furnished them with the necessary variety of ores to stimulate still further the tonnage production of the mines, together with their own growth and prosperity.

Thus the pioneer solution of mining and metallurgical problems has made the rank and file in the State's mining industry the leaders in methods, production, and both mechanical and metallurgical perfection.

The coal product and the transportation facilities of the State have played important parts in its mining history. Its districts have been quickly rendered easy to reach by rail, and the most speedy development has been made possible. At no time has the industry or its metallurgical sister been dependent on resources without the State boundaries. In due course of electrical advance, Colorado was the first State to apply this force to mining operations. The application of electrical power to mining dates back to installations in Aspen, first for mine hoists and later in this camp, as well as generally in most of the other mining districts in the State, to pumps, blowers, air compressors, mine and mill haulage, shops and motors, with the power for these latter applied also directly at the point of its use upon a particular machine. Water-powers in Colorado were developed for electrical conversion and subsequent distribution to the mines. Where such powers are not available, central power-plants are using coal.

In the high, mountainous sections of Southwestern Colorado, the successful operation of a number of large mines is the direct result of the utilization of a more or less distant water-power and its electrical conversion and transmission to the point of desired application in both mine and mill. In such cases solid fuel could not have been transported and used except at such an enormous expense that loss would have been the net result, instead of profit. With the growth

Mr. Hawkins. of particular mining districts, the application of the current has made possible the use of lengthy individual or community development, and transportation tunnels and drifts to cross-cut or open veins at greater depths, with a consequent delivery of the ore product nearer to treatment mills or transportation facilities. It is true that the first few years of the application of electricity to mining showed small progress; but it was entirely due to two factors: The uncertainty naturally felt in the use of a new power, and the time requisite for manufacturers to learn the needs of the industry and become acquainted with the conditions to be met in underground work. Electric drills present the only problem not yet solved by the current with entire satisfaction, but progress in these machines in the past few years has been such that the difficulties, which are altogether of a mechanical nature, will undoubtedly be overcome.

Probably no section of the State, with the exception of the mining camps of Southwestern Colorado, presented as favorable a field as the Cripple Creek District for the use of the electric current in mining operations. There, even though, close to the south, there were large producing fields, the considerable expense of coal furnished ideal conditions for the manufacture and transmission of the current from centrally-located stations at or close to the coal mines. The great monthly ore tonnage of the district from both the larger mines and the many individual lessees, working smaller blocks of mining ground, has made a ready market for electrical power, and its use, particularly for operators of the latter kind, has been indispensable. In no other way could many of the lessees of the district have opened the same amount of profitable ore in the limited time allotted by their lease. In this manner, not only have the normal growth and production of the camp been fostered, but the frequent consequent reduction in the mining cost per ton of ore has actually added a tonnage of ore which could have been made available in no other way. The system of rental of motors has also reduced the initial outlay of the lessee operator, and hence the benefit derived has been shared by the manufacturer of the current as well as the lessee. The Colorado gold production in 1907 was $20 888 833. In 1906 it was $22 934 400, a decrease of $2 045 567 in the last calendar year. Much or all of this decrease was caused by the destruction by fire of the largest ore treatment mill in the State, which was operating entirely on Cripple Creek ores. This calamity necessitated the suspension of contracts for a period of five months. The reconstruction of the mill was completed in January, 1908.

The mining of silver is connected more intimately than gold with ores carrying also the base metal values of lead, zinc or copper, or any association of them. Some of these, indeed a large proportion, either require concentration by putting any possible given number of tons into one, or they become more profitable thereby. Most of the

treatment of this character is applied by the mine at its own plant, Mr. Hawkins. and the product is shipped to smelting centers. In other cases, custom plants either purchase the mined products outright, or charge the shipper at a tonnage rate and return him the concentrated product for his own disposition. The crude and concentrated products, carrying silver and lead values, are reduced to metal at the smelter center. In 1907 the silver product of Colorado was 11 648 136 oz., Troy, having a value of $7 687 769. In 1906 it was 12 447 400 oz., Troy, having a value of $8 185 276. The decline in the price of the metal was sharp during the last five months of the year, in common with the prices of lead, zinc, and copper. The total fluctuation—between 68 and 52 cents per oz.—was marked by a drop of 10.68 cents per oz. from August to November, 1907. A natural curtailment of production resulted, and was increased by a restricted market for lead ores during a portion of the year.

The lead-ore production of Colorado is associated with silver occurrences, and is treated altogether by the smelting establishments. During 1907 the production was 50 000 short tons, having a value of $5 111 000, while the tonnage of 1906 was 51 000 tons, having a value of $5 756 658. During 1907 the price of the metal fell from 6 to 3.65 cents per lb. The drop in prices, resulting finally in the abandonment of any effort to maintain a fixed rate and an open market throughout the country, caused the smelting establishments to discourage the mining and shipment of lead ores for the last six months of 1907, except where contract obligations prevented.

When zinc accompanies lead in silver ores, usually to an extent in excess of 8 or 10%, it frequently renders the ores unprofitable as to their other metal value, and in such cases the electric current, again, has been applied advantageously, and a separation made of the complex ore into three products, namely, zinc sulphide, lead sulphide (both as concentrated products), and the comparatively valueless quartz or rock waste. In the past few years a very large industry has been adding to the State's production—chiefly from Leadville—by this method of separation and concentration. In 1907 the production of zinc from ores was nearly 130 000 tons, having a value of $5 000 000, of which 5 332 tons was spelter, and practically all of it was thus obtained, giving two valuable products from a formerly valueless one.

The copper production of the State is of comparatively little importance, being only a little more than 1% of the whole country's total. The building of new railroads will open virgin copper and gold fields during 1908 and 1909. The production for 1907 and for 1906 has been about 5 000 tons of copper yearly.

The production of metalliferous ores has received, and will always receive, a very large proportion of the benefits and savings arising from the advancements of the metallurgical art. Prosperity, there-

Mr. Hawkins. fore, is directly increased, and, in return, there is mined a greater tonnage of ore. Even though the grade of the average be lower, the net profit is increased; hence the mine, the treatment plant, and the labor employed all share. In a comparatively short time, smelting charges, on much of the ore product of the State, have been reduced, and in many cases rates have been established for the especial purpose of stimulating the production of the fluxing ores, carrying lead and iron in important quantities. These advantages and encouragements have also been extended to shippers of silicious or quartzy ores of the Cripple Creek District, and the tonnage produced has responded accordingly. The mills, however, have been the most important factor in this respect. The chlorination treatment of Cripple Creek ores, first extensively and successfully applied, has been still further improved, chiefly in the cost per ton, by fine grinding and cyanidation of the roasted ores. One large plant is reducing in excess of 20 000 tons monthly from the mines of Cripple Creek. A portion of this quantity is ore having a gross value of less than from \$8 to \$12 per ton, and the mere fact that such low-grade ores return a profit to both the producer and the reducer, where it was formerly impossible to mine them, speaks eloquently of metallurgical progress in this particular field. Ores of still lower value, even such as are being taken in bulk or sorted from what were formerly waste mine dumps, are now being treated in a large way at the new Independence Mill and others. The absence of necessity for roasting these very low-grade ore-dump tonnages reduces the cost so much that it is likely that this practice will be extended to other mines and dumps before the year is over. Metallurgy, therefore, has been a most important adjunct in the progress of a section which still continues to be the largest gold-producing mining camp on the Continent.

The use of the slime treatment and the filter press, both with tube-mill grinding and the cyanidation of gold and silver ores, is yet to come in Colorado, and remains to be developed upon some appropriate ores. The treatment is speedy, the cycle is continuous, and hence the tonnage handled will be so large that the low cost per ton will give an attractive profit. In numerous instances in Clear Creek and Gilpin Counties concentration mills have made important improvements in their mechanical handling and their percentage of saving. In these counties there has been the same value in ore production for each of the past two years, with the advantage, however, that in the past twelve months an unusual amount of development work has been accomplished by the application of the cheaper electrical power transmitted from a central station. As a result of the year's improvements in metallurgical practice, the State will have, from these long-established and substantial mine districts, a largely increased production of both ores and values in gold and silver, as well as in concentrates containing lead and zinc.

The progress of Colorado in a few years in zinc metallurgy has been noteworthy. In spite of the first high cost of plants for magnetic separation, much has been and is being done. Besides the Wetherell separator, which has been the pioneer in this field, the Blake patents, together with important improvements acquired by the Blake Company, have successfully applied the electro-static current to the separation of the sulphides of lead and zinc, and have entered a field not open to the Wetherell separator, but between them they cover all classes of zinc sulphides. Recently, Blake improvements have shown ability to make a closer split or saving than before, and the speedy application of these machines to the general sulphide product from Leadville, Clear Creek, and other districts, may be expected. A. R. Wilfley has devised and made successful the application of a rapid magnetic or magnetizing roasting of sulphide ores, followed by magnetic separation of the product into concentrates and waste. There are in the State already six such plants, built and building, having a total daily capacity of 700 tons of crude ore. Thus the value of the zinc ore and metal product of the State is rapidly approaching that of its lead. Mr. Hawkins.

Still another method, as yet more or less in the experimental stage, however, promises to become a useful adjunct to the mining and successful handling of low-grade lead and zinc sulphides. It is the Elmore process, and is based on the action of dilute acids on crushed low-grade sulphide ores, whereby hydrogen is produced. The gas acts as a float for the sulphide particles, and they are withdrawn by a vacuum suction, dried, and subsequently treated. Enough has been done to indicate eventual commercial success.

It is to be noted with much satisfaction that the leasing system is again being extended in the Cripple Creek District. In the past, this method of development has been most productive of tonnage and value. It is the lessee who makes the most of the new discoveries. All the mining districts of the State need the system. The Northern District, comprising Clear Creek and Gilpin Counties, Leadville, and the Southwestern sections are applying it, to a considerable extent, from all of which material results may certainly be expected.

Gold dredging in Colorado is practically confined to Summit County, on the Blue and Swan Rivers, where two companies are actively at work with several dredges built during the winter of 1907–08.

The great variety of Colorado's mineral production covers nearly all fields of engineering and material needs. For some years past, ores of tungsten have been produced, chiefly in Boulder County. In 1906 the State's product was valued at $225 000, in 1907 it reached double that figure, in spite of the complete cessation of purchases of tungsten concentrates during the last two months of the year. Activity

Mr. Hawkins. in this industry is again established. The largest portion of the country's product of tungsten is mined in Colorado, and the successful development of the new tungsten lamp will create a still larger demand than has heretofore come from the steel industry alone.

Several years ago attempts were made to produce oxides of vanadium and uranium from the sandstone deposits of San Miguel and Montrose Counties, in Colorado. Investigations have shown the presence of these oxides in commercial quantities, and plans are now under consideration for the concentration and shipment of such products to Pittsburg and to Germany. Steel makers are always in the market for these oxides, and the unit price offered is sufficiently high to stimulate production.

The published dividends from metalliferous mining in Colorado were $4 225 000 for the year 1907. The total metalliferous production referred to herein was $40 000 000 in the same year. Other mineral products brought the State's total production up to $94 000 000. Progress thus far in 1908 is at a rate in excess of these totals.

The present railroad construction of the Denver, Northwestern and Pacific is about to enter a section of Northern Colorado from which the State will soon realize increases in gold and copper production as well as an immense addition to its coal output. Central power-plants are under extensive construction in the northern and southern parts of the State, and their completion will mean a great deal to the miner, who will then be able to secure the application of power where the cost of coal has heretofore made operating prohibitive.

No labor troubles disturbed the State, as in former years, and had it not been for the comparatively small effect of the recent financial disturbance on the State's ore and metal productions, felt chiefly during the last six months of 1907, the year would have shown the usual normal gain of 5% or more. The mining wage scale is high, generally, and the miners are in the main satisfied. Industrial peace reigns, and it will not be soon disturbed. Regularity of work has characterized mine operations for several years past, with the exceptions already referred to. The business upheaval during the autumn of 1907 was not felt to any extent. No condition of unusual prosperity had reigned previously, and, while some sister States were keenly affected, Colorado was disturbed very little. It is true, that, as compared with 1906, the gold production fell off 10%, and that the dividend record was slightly reduced, but it is also true that 1907 saw little or no reduction in the number of tons of ore produced, or in the number of hours of employment of the wage earner in mining. What might be called an investment basis in the State's mining operations was reached. Speculation has been largely eliminated, and, from the mining fields, one may confidently look for the regular results to be expected from progressive metallurgical improvement, accuracy, and energy.

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THE MAINTENANCE OF MACADAM AND OTHER ROADS.

An Informal Discussion at the Annual Convention, June 25th, 1908.

By Messrs. Arthur H. Blanchard, John R. Rablin, Louis L. Tribus, James Owen, Nelson P. Lewis, G. N. Houston, D. C. Wedgeworth, Ira O. Baker, and Arthur H. Blanchard.

Arthur H. Blanchard, Assoc. M. Am. Soc. C. E.—Although the subject of this discussion covers the maintenance of various types of surfaces built in the country districts, and hence includes earth, gravel, and macadam roads, this introductory discussion will be devoted exclusively to the surface construction and the maintenance of macadam roads subjected to modern traffic. Mr. Blanchard.

Five years ago the highway engineer had to contend with the action of the elements, poor foundations, narrow tires, heavy loads, and road material deficient either in resistance to abrasion, in hardness, toughness, or in cementing properties. Since then the nature of the traffic to which macadam roads have been subjected has changed materially. The advent of the swiftly-moving motor car has necessitated a readjustment of methods, both of construction and maintenance.

In order to facilitate an intelligent discussion of the subject of the maintenance of macadam roads, from the standpoints of economics and efficiency, it is advisable to consider a classification of suburban and country roads, especially in those States in which a large percentage of the main highways is used by motorists. Otherwise, that important and fundamental problem, the adaptation of macadam roads to modern traffic, will be lost sight of in the discussion of various methods experimented with for the allaying of dust and the preservation of highways.

Mr. Blanchard. It is practicable to recognize three classes of highways existing outside of the city limits, the classification being based on the amount and character of the traffic to which the roads are subjected. Naturally, these classes merge into and overlap each other in many cases, but the classification will suffice as a basis for comparison. The first class includes interstate trunk lines, interurban trunk lines, and popular routes of travel. The second class includes intrastate trunk lines passing through towns, the highways connecting towns situated within a few miles of each other, and secondary streets in towns. The third class includes feeders leading to towns, and highways of the first and second classes from sparsely-populated parts of the country districts, highways connecting towns which are many miles apart, cross-roads, and transverse feeders in towns.

Highways of the first class will undoubtedly be subjected to the heaviest motor-car traffic, although not necessarily to the heaviest commercial traffic. Highways of the second class will probably carry heavy commercial traffic and a limited amount of motor-car traffic. The third class of highways will usually be subjected, in amount, at least, to the ordinary country highway traffic. The maintenance of highways of the third class, which are constructed either of earth, gravel, or broken stone, will not be considered, as it is intended to devote this discussion to the maintenance of those highways of the first and second classes which, being subjected to a more or less heavy motor-car traffic and an augmented commercial traffic, demand an entirely different method of maintenance, and in many cases a modified construction of the broken stone surface. As many macadam streets and park drives within the limits of municipalities are subjected to the same kind and amount of traffic as highways of the first and second classes, the methods of maintenance will be similar. The amount of traffic and the speed attained by motor cars are two of the most important factors to be considered in the discussion of methods for maintaining highways of the first and second classes.

The problem before the highway engineer is to determine what type of road surface for highways of the first and second classes will be most economical from the standpoints of first cost, maintenance, and cost of renewal, and best adapted to the traffic to which it is subjected. In considering methods of construction and maintenance, it is essential to have definitely in mind what is expected to be accomplished by a given mode of procedure. Unfortunately, this point of view has been lost sight of by many who are responsible for the maintenance of urban, suburban, and even country highways. The distinction between the use of various materials as palliatives and dust layers, and the use of the same materials as component parts of more permanent construction is exceedingly important.

Considering the modern methods of maintenance used in the United

States by proceeding from the simple to the complex, the common remedy for a road from which the binder has been removed is to supply the deficiency by spreading an efficient binding material such as sand, gravel, or broken-stone screenings over the middle of the road and allowing traffic to work the binder into the interstices of the top course. This method is usually effective, but is not efficacious for highways of the first class subjected to excessive motor-car traffic. Strenuous objection is naturally made to this mode of procedure by the residents adjacent to the highway, as the dust nuisance is considerably increased. Mr. Blanchard.

In order of simplicity, the methods of retaining the dust and thus aiding in the preservation of the surface should next be mentioned. Almost everywhere, the following methods have been used primarily to allay the dust, although in some cases they have been used for strictly maintenance purposes. Among the methods used in the United States to alleviate the dust nuisance may be mentioned, sprinkling the surface with fresh water, salt water, a solution of calcium chloride, oils with a paraffin base, oils with an asphaltic base, oil of tar, oil emulsion, Westrumite, Dustoline, Asphaltoline, Tarracolio, and deliquescent salts. The methods of painting or spraying the surface with various coal-tar, asphalt, and cement mixtures have been used, both for the preservation of the surface and the elimination of dust. The details of the methods of using the materials mentioned will not be considered in this introductory discussion, except in relation to experimental work in Rhode Island. It is hoped that the practice in other States will be presented by highway engineers who are conversant with maintenance work in the districts which they represent.

Thus far, the methods of constructing more permanent macadam roads have been limited to the use of coal-tar or its preparations, liquid and rock asphalt, and cement as binding materials. Bituminous macadam pavements, of which the modern bitulithic is an example, were first laid in America about 1840. The cost of these pavements prohibits their use except on the streets of cities and important suburbs. Two fundamental methods have been used in the construction of bituminous macadam applicable to country roads. First, to incorporate the binding material with the broken stone by mixing by hand or machinery, and second, to form the macadam surface, loose or rolled as the case may be, and distribute the binding material by dippers or machines over the broken stone, allowing the fluid to penetrate the upper course.

The reconstruction of macadam roads has been accomplished by loosening the surface with the picks of a road roller, or by the use of a scarifier, and afterward impregnating the broken stone with a coal-tar preparation, or with liquid asphalt, or a heavy asphaltic oil.

Mr. Blanchard. In case re-surfacing is necessary, the same method is followed, except that practice differs as to whether or not the new material should be shaped by light rolling before the application of the bituminous binder.

The engineers of France and England have been studying the problem of the adaptation of macadam roads to modern traffic for the past five years, and hence have advanced along some lines of research farther than American engineers. Although many patented solutions have been tried in France and England, most engineers favor the use of a bituminous binder in some form as an integral part of new construction and as a palliative on old macadam roads. The methods, however, vary to a considerable degree. In both countries the introduction of various tar-spreading machines has materially increased the efficiency of the tar-painting method and reduced the cost of applying the tar. The Laissailly machine, used in France, is typical of machines used in both countries. It consists of a mounted tank containing the tar, which is heated by a system of piping through which steam passes. From a tank on the rear of the vehicle, which serves as a reservoir, the tar flows by gravitation out of a series of holes, and is distributed by automatic brushes attached to the body of the vehicle. Devices for applying the tar under pressure form integral parts of other machines, with which it is claimed that a uniform penetration of from 1 to 3 in. can be secured.

In addition to the mixing and penetration methods of constructing tar-macadam roads, English engineers are using the Gladwell system or modifications of this method. This system consists in spreading on the No. 1 course, after rolling, a mixture of tar and chips to a depth of about ¾ in. On this is deposited from 2 to 3 in. of No. 2 stone, which is continually rolled with a light-weight roller until the tar mixture has been drawn up into the interstices of the No. 2 course. The surface is then sealed by painting with tar, after which a thin coat of fine screenings is spread and rolled. In the construction of all kinds of tar-macadam, careful attention is paid to the quality of the material used, particularly with reference to the tar-absorbing power of the material. The experience of English engineers is that iron slag, the basis of Tarmac, stands first in this respect, limestone next, and granite last.

The primary object of the experimental work undertaken in Rhode Island has been the determination of the most economical and efficient method of securing the preservation of the macadam surface.

In the fall of 1906 a section of tar-macadam, 350 ft. in length, was built in Charlestown, the location being on a curve of the interstate trunk line connecting New York, Narragansett Pier, Providence, and Boston. With the exception of the addition of the tar, the method of construction was similar to that used in building an ordinary

macadam road. After the sub-grade had been thoroughly rolled, the No. 1 broken stone, varying in size from 1¼ to 2½ in. in longest dimension, was spread to a depth of 6 in. and rolled to 4 in. Tar, which had been heated to the boiling point in an ordinary tar kettle, was then sprinkled on the rolled surface by using dippers. The No. 2 stone, varying in size from ½ in. to 1¼ in. in longest dimension, was next deposited on dumping boards and thoroughly mixed with hot tar by using rakes and shovels until every stone was completely coated. This mixture was applied on the No. 1 course to a depth of 3 in. and, after the tar had set, was rolled to 2 in. A thin coat of dust, which would pass through a ½-in. mesh was then spread on the surface and forced by rolling into the No. 2 course to fill up the voids and provide a smooth surface. The quantity of tar used was 1.25 gal. per sq. yd. As this stretch of tar-macadam proved efficacious, it was deemed advisable, in 1907, to construct an experimental mile, this section being on the same interstate trunk line as the curve just mentioned. Although it will require from 5 to 10 years to determine the economical status and the efficiency of the tar-macadam road constructed by the method outlined, the perfect state of the surface of the 1906 and 1907 sections in the spring of 1908 influenced the State engineers to advocate the adoption of this method of construction on highways of the first class subjected to excessive motor-car traffic. Mr. Blanchard.

The sections thus far contracted for in 1908 include: First, two sections aggregating 11 300 ft., 14 ft. in width, on the interstate trunk line between Saunderstown and Narragansett Pier, which is subjected to all the motor-car traffic between the Pier and Newport, Providence, and Boston, and the heavy touring-car traffic between New York City and Newport; and second, 11 870 ft., averaging 23 ft. in width, on an interstate trunk line in East Providence, which is subjected to a heavy motor-car traffic and also an abnormal commercial traffic. On one of the sections in the district of Narragansett the State engineers experimented with various bituminous mixtures used in different methods of construction, the object being to determine the most efficient binder and the most economical method of construction which would be adaptable to modern traffic. The original plan was to build, by each method, a 300-ft. section, 14 ft. in width, but the impracticability of certain methods with the tools at hand necessitated a modification of this plan. The experimental work was started after 1 300 ft. of tar-macadam had been constructed in accordance with the method used in 1906 and 1907. Table 1 is a descriptive statement of the various sections.

In the following statements, covering details and cost data of the various sections, it is hoped that sufficient information will be given to enable engineers to use the results of these tests intelligently. It has been characteristic of many descriptive articles dealing with

Mr. Blanchard. maintenance work that the data necessary to render the results acquired valuable to others have been lacking. Although the ideal is impossible of attainment in many cases, it would be advisable if the following points, for example, could be covered in a description of methods of construction of tar-macadam: Class of highway to which the road belongs, and nature of traffic; length and width of road; nature of sub-grade; location of the road relative to the nearest railroad station and source of material; maximum and mean grade of road; kind and size of broken stone composing the surface; months during which road was constructed; maximum, minimum, and mean atmospheric temperature; details of construction; source and analysis of binding material; quantity of binder per square yard; average rate of progress per day, in square yards; specifications covering heating and application of the binder, climatic conditions, etc.; cost per square yard, over and above the cost of ordinary macadam, covering labor, material, accessories, etc.

TABLE 1.

Number of section.	Length, in feet.	Material.	Method, etc.
1	300	Tar-macadam.	Mixing method, with tar omitted from surface of No. 1 course.
2	300	Tar-macadam.	Penetration method.
3	100	Tar-macadam.	Gladwell system.
4	300	Tar-macadam.	Mixing method.
5	130	Tar-pitch-macadam.	Mixing method, with chips added to No. 2 mixture.
6	70	Tar-pitch-macadam.	Mixing method.
7	300	Tar-macadam.	Mixing method.
8	300	Tar-asphalt-macadam.	Mixing method.
9	290	Tar-asphalt-macadam.	Mixing method, with bituminous mixture omitted from No. 1 course.
10	75	Asphalt-macadam.	Mixing method.
11	135	Tar-asphalt-macadam.	Mixing method, without bituminous mixture on surface of No. 1 course.
12	410	Tar-asphalt-macadam.	Mixing method.
13	190	Tarvia-macadam.	Mixing method.
14	172	Tarvia-macadam.	Mixing method, with Tarvia omitted from No. 1 course.

The roadbed selected for the experimental work was on a long tangent, the sub-grade being uniformly built on a 1-ft. fill, and the grade only varying from a minimum of 0.08% to a maximum of 0.76 per cent. The broken stone used was of the same character throughout, consisting of local stone composed of a fine-grained granite and hornblende schist of the same size as that used on the 1906 and 1907 sections. The experimental work was all done in the month of May. The temperature while the work was in progress varied from 60 to 90°, the average being 70 degrees. The tar used was produced at the Providence Gas Works, and cost, f. o. b. Providence, \$2.75 per bbl. The freight from Providence to South Ferry, a distance of 26 miles, distributed over three lines, was \$0.62 per bbl.

The cost of the haul from the South Ferry station to the road, averaging 2 000 ft., was $0.13 per bbl. The barrels cost $0.75 each, which, minus the haul and the freight to Providence, would be refunded by the gas company upon return of the barrels. The net rebate was $0.56 per bbl. The total cost of the tar, therefore, was $3.70 per bbl. Mr. Blanchard.

The coal-tar produced at the Providence Gas-Works is characteristic of gas-house coal-tars produced elsewhere, in that the percentage of water, ammoniacal liquor, light volatile oils, and heavy oils is a variable quantity. In order to determine various facts, including the range of variation, samples, taken directly from the tanks at the gas-works at various times, were analyzed. Table 2 shows the important results of the analyses. The percentage of volatile matter at 120° after heating from 6 to 8 hours represents the percentage which might be lost from the tarred surface of a road during summer seasons. The results in the column headed "Solid matter, in grains per gallon," were obtained by digesting a portion of the sample with an equal volume of water. The object of this analysis was to determine the quantity of matter which might be washed from the surface of a road by a heavy rain. The average specific gravity of the tar was about 1.26.

TABLE 2.—COAL-TAR ANALYSES.

Date.	Percentage of volatile matter at 120 degrees.	Percentage of volatile matter at 260 degrees.	Solid matter, in grains per gallon.
Nov. 20, 1907	10.6	30.7	46.7
Nov. 27, 1907	6.8	27.9	102.2
Dec. 18, 1907	11.1	24.9	105.1
Dec. 24, 1907	7.1	21.3	20.4
Jan. 14, 1908	16.3	32.3	52.5
Jan. 21, 1908	7.2	19.4	5.8
Feb. 19, 1908	5.8	16.2	2.9
Feb. 26, 1908	4.7	15.3	8.6
Mar. 18, 1908	3.1	13.3	2.3
Mar. 24, 1908	3.9	18.1	33.2
Apr. 22, 1908	18.4	29.9	116.4
Apr. 29, 1908	8.0	19.0	5.8
May 20, 1908	7.9	20.6	.....
May 27, 1908	8.1	23.3	.....

In order to determine the time required to drive off a sufficient percentage of the light volatile constituents to render the tar satisfactory for road work, a sample of the tar was heated for certain definite periods, the result of which was as follows:

			Total loss at end of period.
	Loss at 120° after heating	$\frac{1}{2}$ hr. 3.6	3.6
Additional	" " " "	1 " 1.2	4.8
"	" " " "	2 " 0.9	5.7
"	" " " "	3 " 0.7	6.4
"	" " " "	4 " 0.7	7.1
"	" " " "	5 " 0.5	7.6
"	" " " "	6 " 0.5	8.1

Mr. Blanchard. As many failures of tar-macadam have resulted from the use of tar containing a large percentage of light volatile constituents, the State engineers, taking into consideration the foregoing analysis, have decided to require that all tar, before application, be heated for 2 hours at a temperature of not less than 150° fahr., the maximum quantity of tar heated in a 100-gal. kettle to be 50 gal. It will also be required that all water shall be removed from the surface of the tar in the kettle before heating. From one barrel used on the Narragansett section more than 2½ gal. of water were removed.

The pitch cost, f. o. b. Providence, \$3.00 per bbl. of 30 gal. The freight from Providence to South Ferry cost \$0.27 and the haul, averaging 2 000 ft., cost \$0.10 per bbl., making the total cost \$3.37 per bbl. The pitch used was medium hard, having a melting point at 105 to 110° fahr., and a specific gravity of 1.25. The volatilization analysis gave the following percentage results:

Volatile at 120° fahr.	None.
Additional loss at 212°–220°	1.73
" " " 240°–260°	1.53
Total loss " 260°	3.26

A Texas asphalt was used. It was purchased from the Texas Company, and cost, delivered at South Ferry, \$21.50 per ton of 250 gal. The cost of the haul from the station to the road, an average distance of 2 000 ft., was \$0.10 per bbl. of 42 gal. Therefore, the cost of the asphalt on the road was \$0.09 per gal. This asphaltic product is listed by the Texas Company as Texaco Asphalt, Grade H. Analysis yields practically no volatile matter to 260° fahr., a melting point of 140° fahr., and only 8.8 gr. per gal. in a water solution.

The Tarvia was purchased from the Barrett Manufacturing Company, and cost, f. o. b. Boston, Mass., \$3.25 per bbl. The freight to South Ferry, a distance of 70 miles, over three lines, was \$1.26 per bbl. The barrels cost \$0.75 each. The rebate allowed was \$0.65, which, minus the cost of haul, and return freight of \$0.30 per bbl., made a net rebate of \$0.35 per bbl. The cost of the haul to the road, an average distance of 1 000 ft., was \$0.11 per bbl. The total cost was \$5.02 per bbl.

The results of the analysis of Tarvia A were as follows:

Specific gravity	1.182
Volatile at 120° fahr.	2.4
Additional loss at 212°–220°	8.9
" " " 240°–260°	6.5
Total " " 260°	17.8

Having considered the cost and analyses of the materials used, the cost and details of the construction by various methods will be given. Mr. Blanchard.

The tar-macadam constructed by the mixing method required the services of two tar men at $1.75 per day of 10 hours, and three common laborers at $1.50 per day, in addition to the force usually employed on the No. 2 course. The cost of the labor, based on an average rate of progress of 233 sq. yd. per day, was $0.035 per sq. yd. The cost of accessories, which would include rent, or interest and depreciation, on tar kettles, dippers, and cost of fuel, was $0.005 per sq. yd. The saving by not using a watering cart, at $4.00 per day, was $0.013 per sq. yd. The cost of the tar, at 1.25 gal. per sq. yd., was $0.093. The total cost of the tar-macadam, in excess of the ordinary macadam, was $0.12 per sq. yd.

The difference in cost of the tar-macadam without the tar on the No. 1 course and with that tar (about ⅓ gal. per sq. yd.) spread on the No. 1 course, was not appreciable. It is believed that the painting of the No. 1 course is not necessary. In common with all methods of construction, with the single exception of the Gladwell system, it is necessary, in order to secure a maximum penetration of the broken stone by the tar, and adequate incorporation of the tar in the macadam, to allow the No. 2 course to remain without rolling and sanding for from 1 to 3 days, depending on the climatic conditions. It was found to be inadvisable to roll the tarred surface during the warm part of the day, as there was a tendency for the No. 2 course to shift if the tar were soft.

In the construction of the tar-macadam by the penetration method, the tar was spread over the surface by dippers. This method was very unsatisfactory, an unequal application being the result. In order to procure an efficient road, more tar was applied in patching, the original application of 1.25 gal. being thus increased to 1.87 gal. The penetration secured varied from 1 in. to 2½ in. If this method is to be used, pouring pots with fan-shaped spouts, or a fan-nozzle connected with a hose from a tank-wagon, should be used, or preferably a spreading machine similar to the Laissailly or Aiken. Even with a machine of the most approved type, and with the stone heated either before or after deposition, it is doubtful if the tar-macadam surface thus constructed would be as uniformly bound together as when laid by the mixing method. The average rate of progress on this section was 389 sq. yd. per day. The cost of the labor (which consisted of two tar men and one common laborer) was $0.013 per sq. yd. The tar cost $0.138 per sq. yd. Adding the cost of accessories and deducting the rebate due to not watering, the total cost was $0.143 per sq. yd.

In the construction of tar-macadam by the Gladwell system, the bituminous mastic, consisting of tar and stone chips varying in size

Mr. Blanchard. from $\frac{1}{8}$ to $\frac{1}{2}$ in. in their longest dimensions, was mixed in a regular mortar box. This mixture was spread to a depth of $\frac{3}{4}$ in., and the No. 2 course was then laid upon it. A coating of tar was spread on the surface, and, after screenings had been applied, the section was thoroughly rolled. The upward penetration of the tar was not measurable, and the surface coat did not penetrate more than $1\frac{1}{2}$ in. In order to procure satisfactory results, it will be necessary to have the No. 1 course so thoroughly compacted as to hold a semi-fluid mixture; the stone composing the No. 2 course should be larger than that generally used, and should be well heated, and, finally, it will be necessary to use a light asphalt roller in order to draw the fluid mixture gradually to the surface, and not attempt to crush the No. 2 course into the binder. Under no circumstances is it believed that the method will prove as efficacious or economical as either the mixing or penetration methods of construction. The rate of progress of this class of work was slow, and would average 156 sq. yd. per day. The labor item was high, two tar men and four common laborers being required, making the labor cost \$0.06 per sq. yd. The tar, 1 gal. per sq. yd. in the mastic and 1.25 gal. on the surface, cost \$0.167 per sq. yd. Adding the cost of the accessories and deducting the cost of watering gives a total cost of \$0.22 per sq. yd.

Tar-pitch-macadam was constructed with and without using chips mixed with the No. 2 course. With hand-mixing, the omission of the chips is preferred, as the tarred chips have a tendency to coagulate, hence preventing a uniform mixture. The pitch and tar were used in the proportion of 1 to 3. The resulting surface was excellent, and could naturally be rolled and finished more expeditiously than tar-macadam. Under certain circumstances, as for instance, on a road subjected to more or less traffic during construction, this desideratum would be a great advantage. On the other hand, the proportion of heavy oils is less than when only tar is used, thus reducing the elasticity of the surface and increasing the danger of cracking in cold weather. The rate of progress and the cost of labor were the same as for tar-macadam constructed by the mixing method. The total cost was \$0.142 per sq. yd., including \$0.035 for labor, \$0.079 for 1.07 gal. of tar, \$0.036 for 0.32 gal. of pitch, and the usual accessories charge and watering rebate.

In the construction of the tar-asphalt-macadam, 50% tar and 50% asphalt were used, or 0.625 gal. of each per sq. yd. The resulting road was ideal, from the standpoint of construction and as a finished product. A daily average of 233 sq. yd. was built, making the labor item \$0.035 per sq. yd. The tar cost \$0.047 per sq. yd., and the asphalt \$0.056 per sq. yd. The total cost, including the accessories charge of \$0.008 per sq. yd., and deducting the cost of watering, was \$0.133 per sq. yd.

The asphalt-macadam, considered from an economical standpoint, was not successful. The primary difficulty was that it was impossible to obtain a thorough mixture of the asphalt and the stone, coagulation taking place at once. The resulting surface was entirely satisfactory, and could probably be built economically by using a suitable mixing machine and heating the stone. The labor and material items on this work were excessive, the cost of labor being $0.083 per sq. yd., while the cost of the 3.59 gal. of asphalt used per square yard was $0.323. The average rate of progress per day was only 117 sq. yd. The accessories charge was $0.008, and the watering rebate $0.013, making a total cost of $0.401 per sq. yd. Mr. Blanchard.

The Tarvia-macadam constructed by the mixing method appeared to be a *fac-simile* of the tar-macadam made with tar distilled for 3 hours on the road. It is believed that it is primarily a question of economics whether it is preferable to take gas-house coal-tar direct from the works and distil it on the road, or purchase distilled coal-tar, in the form of Tarvia, for example. It should be borne in mind, also, that tar distilled at permanent works will give a more uniform product. The rate of progress was the same as with tar-macadam, hence all cost items were identical, with the exception of the cost of 1.25 gal. of Tarvia used per square yard which was $0.124. The total cost of the Tarvia-macadam, therefore, was $0.151 per sq. yd.

The preservation of the surface of existing macadam roads has been attempted in Rhode Island by using the tar-painting method, and by the use of various oils as palliatives. One application of the tar painting has been used successfully in the preservation of the macadam surface for two years on a hill having a maximum grade of 7.28%, but, where it has been used on roads subjected to high-speeding motor-cars, parts of the coat of tar have invariably peeled off during the first season. On a town road in Peace Dale, painted with hot Tarvia by the usual method in August, 1907, barely 60% of the tar coat was intact in March, 1908. The experience of France, England, and America with the tar-painting method has led many to the rational conclusion that it is not effective for macadam roads subjected to even normal motor-car traffic if high speeding is allowable.

Although the method has not yet been used in Rhode Island, the speaker believes that the most satisfactory and economical treatment of existing macadam roads is to tear up the surface to a depth of from 3 to 4 in. with a scarifier, re-shape the surface, and then rebuild as a tar-macadam road, using the penetration method of construction. On 1908 work in Rhode Island the cost of scarifying was $0.007 per sq. yd. The cost of re-shaping, without the addition of new road metal, would be about $0.005 per sq. yd.; hence the total cost, based on the cost of the penetration method used on the Narragansett Road, would be $0.105 per sq. yd. If a tar-spreading machine were used, the cost, without doubt, would be reduced from 15 to 25 per cent.

Mr. Blanchard. The oil products used on the highways of Rhode Island include Dustoline, petroleum residuum, Ragland crude oil, petroleum fuel oil, and Texas asphaltic road oil. The average cost of oiling roads in Rhode Island, using about ⅓ gal. per sq. yd., has been approximately \$0.01 per sq. yd. Experience would indicate that oiled roads, if efficient, should be treated at least three times during a season of eight months.

The speaker's conclusions, based on the maintenance work he has outlined, and the experience of French, English, and American engineers, are: First, that highways subjected to heavy high-speed motor-car traffic should be built with a bituminous macadam surface constructed by the mixing method; second, that existing macadam roads subjected to a similar traffic should be reconstructed as bituminous macadam roads using the penetration method, or, if re-surfacing with new road metal is required, by using the mixing method; and third, that the economical and efficient treatment of macadam roads subjected to a moderate amount of motor-car traffic is at present a matter of conjecture, requiring for elucidation the acquisition of reliable detailed information with reference to the use of the various palliatives now on the market.

Mr. Rablin. JOHN R. RABLIN, Esq.*—The roads built by the Metropolitan Park Commission of Massachusetts have been constructed from time to time between 1897 and the present, so that none is more than 10 years old. Heavy traffic is excluded from them, and only light pleasure travel allowed. Under these conditions, and with ordinary care, these roads would have been expected to require no extensive repairs up to the present, and they were generally in excellent condition up to the spring of 1906.

At about that time the increasing use of the automobile became noticeable, and in July, 1906, these roads began to show the effects of this traffic. The macadam roads were stripped of their top surfaces, leaving the stone of the base entirely bare, and with no binder to hold it, the stone loosened, and the macadam began to break up.

Investigation was at once begun to find a method of protecting the surfaces, and it was decided to treat, with a specially prepared coal-tar, portions of the roads showing the greatest damage. Consequently, in August and September, 1906, about 3½ miles, or 70 000 sq. yd., of macadam roadway were treated with this material, as an experiment.

The process of treatment was: First, all loose material was cleaned from the surface by sweeping with street sweepers and push-brooms, so that the tar might penetrate properly. The tar was then flowed upon the surface from tank wagons, in which it was brought from the works, at a temperature of about 200° fahr. It was spread with push-brooms, and allowed to set from 4 to 6 hours before covering. Stone

* Engineer, Metropolitan Park Commission, Boston, Mass.

screenings were then spread over the tarred surface in such quantities as were required to absorb the surplus tar which had not penetrated the macadam, and the road was then rolled once or twice, and was immediately opened to travel. Mr. Rablin.

The cost of the work done in 1906, including all labor and material, averaged 6¾ cents per sq. yd.

The results of the treatment, during the remainder of the year, were excellent, a new surface having been provided which could not be removed by the automobile travel, and the road was rendered practically dustless and entirely free from mud. No sprinkling with water was necessary.

This work passed through a very severe winter and, except a small section on which the sub-grade was poor, was in very good condition in the spring of 1907.

In July and August, 1907, it was necessary to re-treat about one-half of the 3½ miles treated in 1906, and to patch the other half. The average cost of maintenance of the whole 3½ miles of roadway, for 1907, was 3½ cents per sq. yd. No other repairs or sprinkling with water were necessary during the year.

During 1907, in addition to the maintenance of the work laid in 1906, various pieces of road, aggregating about 90 000 sq. yd. of roadway surface, were treated with this same material, at an average cost of 7.3 cents per sq. yd.

At the present time, June, 1908, work is in progress on repairs and re-treatment of the surfaces previously treated, and it seems evident that it will be necessary practically to re-treat once each year in order to maintain this work in good condition. Compared with the cost of re-surfacing the roadway with macadam, this cost is small, and the benefits are considerably greater.

From observation it appears that the durability of this treatment is much greater and its life longer on steep grades and on roads with a good, well-drained sub-grade.

Generally, the results of the treatment of the parkway roads with this material have been satisfactory, and it has been very effective in the preservation of their surfaces and in making them practically dustless and free from mud. The treated surface has an appearance similar to tar-concrete, but is not as smooth. No odor is apparent after the material has been on the road a few hours, and what little there is at the time of application is not at all disagreeable.

Various experiments were also made in 1907 in the use of oil on the parkway roads. About 1 mile of gravel-surfaced road and ¾ mile of macadam-surfaced road were treated with a product of crude asphalt oil, from which the naphtha and other volatile substances had been removed, leaving the heavy base material of the oil with sufficient petrolene to make it fluid. This material was applied to the roads,

Mr. Rablin. without heating, and was allowed from 4 to 7 days for its proper penetration. Where surplus oil collected in any slight depressions or ruts in the road, enough sand or gravel was used to absorb it.

This treatment requires a road to be in particularly good condition, either new or newly re-surfaced, and very little cleaning is necessary, only the removal of loose dust.

These roads were treated with the oil in June, 1907, and have remained in excellent condition to the present time, a period of about 1 year. With some slight re-touching, and patching with the same material, arrangements for which are now being made, it is expected that these sections of roadway will remain in good condition for another year before requiring an entire re-treatment.

The cost of this work was 6 cents per sq. yd. for the material and its application, and the cost of maintenance for this year will probably be less than one-half that amount.

It has proved an effective treatment for both macadam and gravel-surfaced roads, and arrangements have been made for the treatment of about 4 miles, or 70 000 sq. yd. additional.

In 1907, about 12 miles, or 240 000 sq. yd., of macadam and gravel-surfaced parkway roads were treated with a mixture of water-gas tar and oil similar to that above described. This mixture was made in varying proportions of from 2 bbl. of oil and 6 bbl. of water-gas tar, to 4 of oil and 6 of water-gas tar. The greater the proportion of oil used, the longer the durability of the treatment.

This treatment is a dust layer, but is not particularly effective in the preservation or bonding of the road surfaces. It is, nevertheless, a good and economical treatment for gravel road surfaces where the bonding effects are not as necessary as upon macadam. It is most deserving of consideration on account of its cost, which is from 3 to 3½ cents per sq. yd., and one treatment is practically effective for laying the dust for a whole season from May to November.

One other experiment is being tried at the present time, and that is the use, on macadam road surfaces which are in fairly good condition, of a mixture of water-gas tar and coal-tar in about equal parts. The use of water-gas tar in this and the previously described mixture, with oil, is for the purpose of diluting the heavier and more expensive material, and thereby reducing the cost per square yard for treatment, and also on account of its penetrating power. No cleaning is necessary for this latter treatment, except where an excessive amount of loose material may have collected upon the sides of the roadway. One or two days after application the surface is sanded lightly and the road opened to travel. The cost of this treatment is about 4½ cents per sq. yd.

It appears to be giving excellent results, and will doubtless prove very worthy of consideration for the preservation of roads which are in fairly good condition.

It has become absolutely necessary to adopt some one of the various forms of treatment to maintain macadam and other roads in fair condition where they are subjected to heavy automobile travel, and, although there are doubtless other, and possibly better, methods, those herein described have been proved to have considerable merit, particularly if applied to the road conditions to which they are best adapted. Mr. Rablin.

LOUIS L. TRIBUS, M. AM. SOC. C. E. (by letter).—In the United States comparatively little attention has been given in the past to the proper maintenance of street pavements. Much attention has been given to the questions of construction, through all their features of subsoil, grade, drains, character of foundation, surface, cross and longitudinal slopes, relationship of pavement surface to traffic in reference to dust, cleanliness, slipperiness, durability, ease of repair, and cost of construction and maintenance. In general, cities have been content to make repairs when road surfaces have, through months or years of use and abuse, developed ruts or holes and a generally unsightly and even dangerous condition. This is not always the fault of the officials in direct charge, but of those who hold the purse strings, who are disinclined to appropriate money to maintain something that seems to be in fair order as far as hasty inspection goes. Mr. Tribus.

There can be no doubt in the minds of municipal engineers that the keeping of any form of pavement in perfect condition adds materially to its total life, and by adding together all costs of such maintenance repairs, for the normal reasonable life of the pavement, the total will not equal the costs of practically reconstructive repair, at longer intervals. If, in addition, there could be tabulated the expense in broken axles, strained tendons, paint and varnish injured by mud, clothing damaged, wear and tear on the nerves, profanity induced, etc., the balance would be very largely on the side of daily maintenance.

In that portion of New York City where the writer has more or less authority, there was inaugurated, some years ago, the system of daily inspection of all the more heavily traveled roads, covering practically all classes of pavement, macadam, brick, asphalt block, sheet asphalt, wooden block, bituminous concrete, iron slag, granite, etc. In rainy weather, when depressions show most conspicuously, the men stationed on macadam roads, note the depressions and, if possible, at once clean them out and fill them in with new broken stone, of such assorted sizes as need may require; a little hand tamping, given at the time, finishes the repair, and a few days' traffic makes it indistinguishable from other parts of the roadway. When a road wears down so that the aggregate cost of daily repairing exceeds the economical limit, then reconstruction takes place.

What has just been said, of course, obtains more particularly for such a pavement as macadam than for the more permanent forms,

Mr. Tribus. though a neglected rut or an improperly made repair in any pavement, after opening the street for some purpose, not only causes an unsightly and even dangerous condition, but seriously injures the adjoining portions of the pavement and tends to lessen the life of the whole.

It has been the aim during these past years to make these repairs permanently and perfectly at the first day possible after an opening is made and back-filled. While such repairs can be more readily made in some classes of pavement than in others, the general principle involved is the same, and the result is almost as satisfactory in one as in the other.

Cleanliness is another very important factor in the life of a road of any class. A macadam road kept reasonably free from dust means freedom from mud; consequently, it saves annoyance, and the stone is held together longer and more effectively than is the case where a loose medium, like sand or mud, aids in breaking the bond and separating the stones.

The writer remembers a long stretch of macadam road, which actually wore down until the tops of the telford foundation stones showed, without any raveling of the surface, due entirely, in his opinion, to the freedom from dust and mud, the traffic being very heavy most of the time.

A sheet-asphalt pavement kept clean will last longer than when left in a dirty condition, as in the latter case water does not flow freely from the surface, and thus tends to rot the asphalt. If water can be kept from standing on the surface, the wear will come from traffic only. A block pavement kept clean is thereby freed from a great many disintegrating elements that find their way between the joints and tend to wear the blocks on their edges as well as on their upper surfaces.

It is a fair proposition, therefore, to say that a comparatively inexpensive pavement, well laid and kept clean and in constant repair, will outlast any more costly pavement that is neglected; and during their respective lifetimes one will give public satisfaction and the other will be a nuisance during all its later life.

The writer has only taken up this question of the cleaning of pavements as it affects the life of the pavement, entirely apart from the sanitary side and the general comfort of the community.

There are so many conditions affecting the selection of the form of pavement for any given street or locality, that it would make a lengthy discussion to go into that topic in detail.

Naturally, the first consideration is the ability of the community to pay for what are its best interests. Assuming, therefore, that the first cost is not limited, a pavement should be selected that will give the least noise, the smoothest surface, and may be cleaned to the best

advantage, all consistent with the weight of traffic, and the traction power of horses, determined largely by the gradient. Mr. Tribus.

With these general questions settled, then come the estimates for annual repairs; the total life of the pavement under the traffic estimated; inconvenience to the public by having the street out of commission for renewal, etc.

These conditions might readily indicate, therefore, the need for a more expensive and more permanent form than the first considerations would suggest; so that no general rule can be laid down for the original selection of a pavement. The principles of construction are standard, but the practice as to maintenance is far from being standardized; and the writer hopes such discussions as this may result in securing more continued and constant cleaning and repairing of all classes of pavement than is as yet customary in the United States, and he knows that the results will tend toward economy and comfort.

James Owen, M. Am. Soc. C. E.—The Society and the country at large should be congratulated on the excellent discussion presented by Mr. Blanchard. It is excellent in its arrangement, and in the description of work done. It also gives tangible results, for it shows the important item of cost, which should be impressed upon every engineer engaged in highway work so that his clients may have a basis of comparison in connection with the change which is about to come in road construction in America. Mr. Blanchard shows that the increased cost over old methods is 12 cents per sq. yd., and if that is impressed upon the people's minds, the small extra expense of building a road according to modern ideas and regulations will be quickly appreciated. Mr. Owen.

After much experience in road building, the speaker is compelled to confess that he must apologize for the work he has done. History sometimes repeats itself, and it is a fact that about 1843, after the good roads era had been fairly completed in England, and they had the system which has been handed down, the construction of those good roads created a desire for self-propelled vehicles. They were not called automobiles then, but were steam carriages. They became so prevalent that the horsemen, and stage drivers, and all other persons not interested in that mode of transit, protested so vehemently that Parliament passed a bill abolishing the steam traffic on the highways of the country. This was really prior to the railroad era. After the lapse of a good many years, the self-propelled vehicle has come again, in a different form, and has come to stay, so that the engineer must now accept such means of locomotion as a fact and must make due provision therefor.

One point should be brought out here, namely, that, nowadays, the term "hard roads" should be eliminated. Hard roads, according to the popular idea, are not wanted. The later road developments show that a medium is being placed in the hard material to obviate the hardness,

Mr. Owen. the abrasion, and the extreme wear, tear, and dust of those surfaces. The necessities for the surface to-day are what would be called pliability and resilience, a surface that will give and be elastic, and will be restored normally after it has done its work. It is proposed to introduce certain foreign material in order to encourage that resilience and elasticity. Two extreme cases may be cited to exemplify that idea. In California earth roads were covered with shale, and then sprinkled with oil, which gave a perfectly smooth surface and the desired degree of elasticity that prevents wear. The second extreme case is the ideal pavement made with asphalt. Both these surfaces are elastic, and do not wear out. Some years ago, the speaker asked one of the asphalt representatives how much wear there was on the surface of asphalt pavements. He replied that there was no wear. Careful examination shows that, practically, this is a fact. One section of a city road under the speaker's control, originally covered with macadam, had to be repaired, on account of travel, every two years, with a coating of about 6 in. of broken stone. Asphalt was substituted, and has been down about seven years, and there has been not a particle of wear, nor a movement of the surface in that time.

This expresses the speaker's idea of an elastic surface. If there is a perfect surface of this kind, the element of wear is almost eliminated. Furthermore, having that elastic surface, the question of the positive texture of the stone used is not of such great importance. New Jersey has been priding itself somewhat upon its roads. They are good because, for the system of construction in use, the material is good. The difficulty, in the different States and sections of the country, is in finding material with which to make good roads; thus this question of mixture is important, and may decrease the trouble of obtaining for road surfaces such good material as was formerly required. The suburban districts of New York City have had the same experience in the treatment of roads as that related by Mr. Blanchard. Automobile traffic has affected the roads to some extent, and some method must be contrived to prevent the resultant damage. The trouble is similar to that in Rhode Island, and Mr. Blanchard has gone into the matter in great detail, and has given tangible information, and definite results.

The use of oil in the vicinity of New York City has not met with favor. It certainly eliminates the dust, but, in residence districts, the odor of the oil is very objectionable, and the oil dust that rises under certain conditions is injurious to everything it touches; consequently the results indicate that tar, or preparations of tar and asphalt, will be used finally to decrease the wear of the roads.

Three years ago the speaker re-coated an old surface with 3 in. of broken stone and tar, with the screenings on top, the cost being about 17 cents per sq. yd. The result was most satisfactory, the road having been neither sprinkled nor repaired since that time.

It seems proper here to allude to roads which are not within congested districts. Professor Baker, who is very enthusiastic about the improvement of roads in stoneless districts, where the necessity for the proper administration of the business of farmers is absolute, considers that the elimination of the necessity for extremely hard material is going to be an important factor in the future construction and development of roads in stoneless countries, like Nebraska, for instance. Mr. Owen.

The oiling of roads, without the intervention of other materials, seems to give satisfaction in certain localities. The plan of using inferior material and a binder with it, is one on which there has not yet been much experimentation, and the suggestion is made that, if possible, this matter be taken up for discussion, so that, in the development of roads for the Central States, some advantages may be gained.

NELSON P. LEWIS, M. AM. SOC. C. E.—In the past the highway engineers of the United States have been disposed to confine their studies, investigations, and discussions to what have been called permanent pavements. By permanent pavements is meant those which are usually laid under contracts calling for a guaranty for maintenance for a year or a term of years, and in which the problem of maintaining and repairing is not assumed by the municipality or State almost immediately upon their completion. Mr. Lewis.

There has been a disposition to regard macadam pavement as a temporary expedient, in a city where the abutting property is not able to bear the cost of a more permanent road material, or as a mitigation of the discomfort of the unimproved suburban or country highway. Only in a few localities where there are numerous towns and villages have macadam roads been built and maintained so as to furnish highways upon which high-speed motor vehicles can be driven with any degree of comfort. It follows that the problems of constructing and maintaining roads of the macadam type have not been the subject of careful investigation by engineers, and the results of the work which they have done have not been presented in the careful manner which is demanded by the importance of the problem.

The engineering profession is greatly indebted to Mr. Blanchard for having placed before its members so full a description of the work of the Rhode Island State Board of Public Roads, and such a complete analysis of the cost.

Lately, special attention has been drawn to the construction and maintenance of such highways by the advent and general use of motor cars. As these vehicles pass along what have formerly been good highways, they are followed by great clouds of dust, resulting in much discomfort to others using the highway and to those living in its immediate vicinity. A resident of Northern New Jersey recently said to the speaker that, while formerly one who wished a country home could purchase property 200 or 300 ft. in depth, place his house 100 ft.

Mr. Lewis. from the highway, and live in comfort, this has now become impossible. To avoid the dust raised by motor cars it is now necessary to locate one's dwelling 500 or 600 ft. from the highway.

The dust raised by motor vehicles, however, means more than physical discomfort. It means that the road material is being used up and wasted with great rapidity. The ordinary city pavement, especially bituminous pavement, and possibly macadam, if kept in good surface, may be improved by the passage over it of heavy vehicles moving at moderate speed; and, within the limits of cities and incorporated towns, the speed of such vehicles is usually kept within reasonable limits by police regulation; but when automobiles, at speeds of from 25 to 50 miles an hour, pass over these highways, with broad rubber tires drawing the binding material from the road metal, the case is very different.

If these highways are not to be entirely destroyed, the speed of these vehicles must be kept within reasonable limits, and some method of construction must be devised which will result in a surface compacted with a binding material of such a character that it will not be drawn from between the pieces of road metal and carried away as dust, or special roads must be provided for high-speed motor traffic. If the latter plan were adopted, such highways could not be built at public expense, but would be the result of private enterprise, and the expense of construction and maintenance would be met by tolls paid by those using them. This plan has lately been adopted on Long Island, N. Y., where a motor parkway is being constructed for the exclusive use of high-speed motor vehicles. This parkway will be entirely separate from other highways, there being no grade crossings, and it is expected that the enterprise will be supported by the fees charged for its use.

To control the speed of motor vehicles on country roads is well-nigh impossible, and in most cases the only treatment which appears to be possible is that of a method of road construction and maintenance which will adapt the highways to the new conditions. This is the plan which is described by Mr. Blanchard. The objection which has been thought almost fatal to such a plan has been the greatly increased cost of construction, but the data submitted by Mr. Blanchard show, in a manner almost startling, that this increased cost, under an intelligent plan and competent supervision, would be relatively little.

Roads built with bituminous cementing materials may present exceptional difficulties in making emergency repairs. Breaks or holes in the surface will inevitably occur, either through a defective spot in the foundation or from some external cause. To repair such breaks, either by the mixing or penetration process, would be difficult and expensive under any system of maintenance, and the members of the Society would doubtless be glad to know whether Mr. Blanchard has formulated a plan for making such repairs. He states that the differ-

ence in cost between ordinary macadam pavement and the bituminous macadam described by him has been as little as 12 cents per sq. yd. This is certainly a remarkable record, and the cost of labor must be very low and its efficiency very high in Rhode Island. The speaker is under the impression that this slight difference in cost could scarcely be approximated in a typical American city, especially where the prevailing rate of wages and the minimum length of day are religiously observed. Still, the ratio between the ordinary contract price and the increased cost of the kind of construction described should be realized if equally efficient supervision and careful attention to detail could be insured. Mr. Lewis.

Mr. Owen has spoken of the desirability of elasticity and resiliency in a roadway for motor cars, and has referred to the earth road as a type of the desirable highway. The speaker confesses his lack of faith in these qualities as desirable for an automobile highway for high-speed cars. At low speeds the earth roadway is admirable, but the speed at which cars run at the present day would seem to require a very hard and unyielding surface.

That the problem presented by the general use of high-speed motor vehicles has arrested the attention of the most painstaking highway engineers of the world is shown by the fact that the French Government has arranged for an International Highway Congress to be held in Paris during the present year for the express purpose of studying these problems. The French highways have been among the finest in the world, and it is stated that they are fast being destroyed by high-speed motor cars. The Congress is to consider, not only the problems of construction and maintenance of the roads, but that of cars and tires as well, with the use of chains and other devices to prevent skidding.

The speaker has had the privilege of examining the Rhode Island roads with Mr. Blanchard, and has been greatly impressed with the results which have been secured. The surfaces of the bituminous macadam roads, constructed as he has described them, are practically perfect, although some of the roads have been in use for two years, and they are absolutely dustless. Portions of them are in sparsely settled localities where there is apparently no restriction upon the speed of motor vehicles, and motor cars pass along the roads swiftly and smoothly without the customary trail of dust, which, however, is emphatically and offensively apparent the moment the cars pass from the bituminous macadam to that of the ordinary type. In the speaker's judgment, Mr. Blanchard has made notable progress in solving what is probably the most serious problem confronting highway engineers at the present time.

G. N. HOUSTON, ASSOC. M. AM. SOC. C. E.—The speaker had the good fortune to live during the first twenty years of his life in Mr. Houston.

Mr. Houston. northern New Jersey, the section referred to by Mr. Owen, where excellent macadam and telford roads have been constructed from the crushed trap rock of the Orange Mountain. He is also somewhat familiar with the conditions in Illinois referred to by Professor Baker.

In Colorado many varying conditions are encountered. In the eastern part are the agricultural districts, where the farmer must be provided with roads over which to haul his produce to the nearest market or shipping point. Of this traffic, the most wearing on the roads is that caused by the hauling of sugar beets, the loads weighing from 2 to 4 tons.

In that part of the State good material for ballast is either lacking entirely or too expensive to obtain. On one of these roads, in the southeastern part of the State, the only material available was what appeared to be a fairly tough shale. On excavating to a depth of 12 in., however, it was found that the material below this point was only partly consolidated, being damp and pliable. On being exposed to the air for several days, it became quite hard, and similar to that on the surface. There being only a limited quantity of surface shale, it was decided to use the softer material below. This was excavated with slips, and placed on the roadway, 8 in. thick at the center and 2 in. thick at the sides, the roadbed being 15 ft. wide. It was then rolled to the proper cross-section with a 10-ton roller, and, after hardening, proved to be a satisfactory ballast. The cost, without the rolling, was 29 cents per lin. ft. of roadway.

In other parts of the State there is the problem of a very high water-table. In some parts of the San Luis Valley it is within 6 in. of the surface. Much of the soil there is a heavy adobe which absorbs this water by capillary action, freezes in the winter, and when thawing heaves the roadbed from 12 to 18 in. above grade. The only remedy for this seems to be to ditch the road on each side, using the material to grade the roadway from 18 in. to 2 ft. above the surrounding country.

In the mountains there is usually excellent ballast, the best being a disintegrated granite. This resembles a finely crushed stone, being angular, and varying from $\frac{3}{4}$ to $\frac{1}{4}$ in. in greatest dimension. Some binder is usually needed with it.

When a mountain road in Colorado is properly constructed, it needs comparatively little maintenance, except when it is misused, as is often the case in the mining districts. For instance, a mining camp opens up, and roads must be built to get the ore out and supplies in. Not knowing whether or not it will develop into a permanent camp, the work is more or less temporary in character. This is more especially true of the bridges, which are usually rough log affairs. If a bridge goes out on account of high water, or becomes too rotten to

FIG. 1.—TYPICAL REINFORCED CONCRETE BRIDGE FOR COLORADO ROADS.

FIG. 2.—PART OF THE RED MOUNTAIN ROAD FROM OURAY TO SILVERTON, COLORADO.

carry the loaded wagons, and the district has not developed sufficiently to warrant a new bridge, the miners then begin to pack in and out, and it takes but a few trips of burros loaded with timber or rails to destroy the remainder of the road. Great damage to mountain roads is also caused by driving herds of cattle over them. Mr. Houston.

The Legislature of Colorado appropriates about $150 000 every two years for the construction of roads and bridges. This is used in making new roads, improving existing ones, and building bridges.

The grades exceed 20% on some of the old roads. On all new work under the State the maximum grade is limited to 12%, with a 6% maximum adverse grade. In the mountains the width varies from 8 to 14 ft., with a 16-ft. width at turnouts.

The law requires that the county in which a road or bridge is built by the State shall maintain it, so that the State, officially, has very little to do with keeping up the roads. In order to reduce this maintenance to a minimum, reinforced concrete bridges are built wherever practicable.

The State has completed, or has under contract, twelve of these bridges, varying from 462 ft. long (eleven 42-ft. spans) to a single-span, 125-ft. arch bridge. Most of these bridges, however, have been of the slab and girder type (maximum 50-ft. span, 16-ft. clear roadway), and designed for a 16-ton traction engine loading. These cost from $42 to $45 per lin. ft., including pile foundations.

Owing to the dry climate, the roads in Colorado are in excellent condition during 10 months of the year, and are seldom impassable on account of mud during the remaining 2 months. Owing to this fact, there is an increasing demand that grades be cut down to accommodate the growing automobile traffic.

During the present year (1908) the State has taken up the experiment of convict labor on road building. A road has been planned through the State, extending from the northern to the southern boundary, and the convicts are now working at the southern end. Reinforced concrete culverts and bridges are being built entirely by this labor.

The design and supervision of construction of all State roads or bridges is under the State Engineer.

D. C. Wedgeworth, Assoc. M. Am. Soc. C. E. (by letter.)—The importance of highway improvements depends not so much on its engineering elements, as on the fact that a real improvement adds a certain percentage to the value of the property it affects. The rural districts are beginning to understand that the slight increase in taxes, if spent on properly improved highways, is a good investment. Mr. Wedgeworth.

This work of improvement has been entrusted to the engineer, and it is safe to say that it has not proved a very simple problem. The usual method is to lay a macadam roadway. This has been to a large extent successful, but the writer believes that a slight alteration

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of the method will add to the life of the road and lessen the cost of construction as well as maintenance.

This cost is dependent on many things, mainly the source of supply of material, stone, filler, water, etc., and, unfortunately, the cost of improvement of a highway is a large factor in its treatment. Against increased cost may be balanced a cheaper construction with probably a poorer highway. For this reason the engineer must strike a fine balance between an ideal roadway at high cost and an inferior roadway at less cost. In arriving at a final result there should be no better criterion than the results obtained from methods and materials heretofore used.

Given a road having a porous yet stable soil, well drained, it is frequently found that a slight improvement of natural conditions affords a good roadway. The worst natural conditions require the best road, the most thought, and frequently the most expense. Under such conditions an ideal road may be prohibited by great expense.

The engineer is then forced to find the material and method of construction which will give the best road for the money to be expended; or, in other terms, the limit of expenditure is determined by the value of the results obtained per dollar of expenditure.

The basic principle of a macadam road is the placing of a suitable wearing surface over a solid base in such a way that the macadam will perform two functions, first, sustain the wear of traffic, and second, transmit to the base the loads imposed by the traffic.

Had the engineer always at hand—or the money to bring to hand—materials which would furnish the macadam capable of carrying out these functions, the problem would be more simple; but, to bring about these results with the materials at hand, calls for much thought, experience, and common sense.

Although the matter of general location and sub-base of a roadway is very important, it does not enter into this presentation, as the writer wishes to deal only with the construction of the macadam proper. Having settled upon the location and prepared the base, or sub-base, as the case may be, the present method has for its object the formation of a 6-in. layer of crushed stone, bonded and placed in such a way as to form a solid mass impervious to water. To this end a stone of good binding qualities must be found. This must be bonded with a proper filler, and the whole consolidated by successive manipulations until this result is approached as nearly as may be. The roadway is then put on trial; if it stands, the material and manipulations have been correct; if not, reasons and repairs are in order. Considering the varieties of stone and filler at hand, differing with various roads, it is rather remarkable that so large a percentage of roads shows satisfactory results.

The point the writer wishes to bring out is this: The stability of

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the macadam is dependent on the bond, and this is dependent on the filler and the manipulation. Under the best conditions, there are seasons of the year when this bond is temporarily destroyed by the action of freezing and thawing. Heavy loads, also, may destroy the bond before the load is transmitted to the base. This fact is shown by the early repairs necessary on roads built under the present method. It is a fact, also, that when a country road is macadamized, not only will the number of vehicles be increased, but the weight of single loads will be doubled.

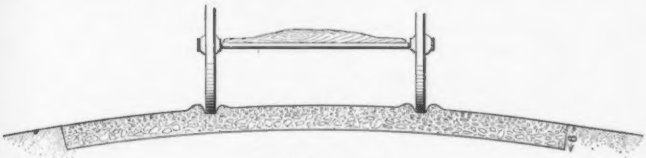

FIG. 1.

Fig. 1 illustrates this point. The stones are pushed up on each side of the wheel track, and not down into the base, simply because the bond is not sufficient to hold them together. A close examination during repair work will show that this is exactly the case.

Proposed Method.—Would it not be a more logical and common-sense way to place the stones in the macadam before crushing, as far as possible, than to break them up and try to bind them together again? Does a carpenter cut a post in sections with the intention of splicing it again when in the building? Why not save this expense of crushing and rolling as much as may be?

FIG. 2.

Fig. 2 represents a section built after the following method: Having prepared the base in the usual manner, the larger stones are hauled upon the base and placed near one another, no special care being taken, as that would take much time and cause expense. These stones, in their vertical dimension, should average about 7 in. for an 8-in. thickness of macadam. Smaller stones, from the crusher, or spawls from the quarry, are then drawn in, and the spaces between the large stones well filled. A wearing surface of about 2 in. is placed over all, and the whole well rolled. As much stone dust as the stone will take up is then applied, and the surface is puddled, as in the present method.

The large stones transmit the load directly to the base. Filler is not depended on to bind the whole together, as the smaller stones

Mr. Wedgeworth. between the large ones will do this. In fact, very little filler will reach the bottom, thus leaving it open for drainage, and avoiding the danger of disintegration by frost.

Comparative Cost.—To make an accurate comparison between the cost of the present and the proposed methods, similar conditions must be imposed on each. On roads where local field stone can be used without crushing, and crushed stone can be imported at reasonable rates, the percentage saved will be greater than where all stone must be hauled from the quarry in either case. Whatever the conditions, there is a saving of 50% of the crushing and 40% of the rolling in comparison with the present two-course road. Another point not to be overlooked is the utilization of the total product of the crusher. This method takes out the dust and 1-in. to 2-in. stone, putting all the remaining product in one bin. This product should be used to fill the spaces between the large stones, and the 1-in. to 2-in. stone for the wearing surface. In case a local stone is used for the body of the road, and trap is imported for the top, very little crushing on the ground will be necessary, and the whole product, with the exception of the dust, may be put in one bin and used to fill the spaces between the large stones.

On an average road, with field stone convenient to the roadway, and a good limestone quarry along the road, a comparison of cost would be somewhat as follows:

Cost per 100 ft. of 12-ft. Roadway.

			Present method.	Proposed method.
Preparation of base, say average of 6 in. excavation or fill, with necessary rolling and grading	22.2 cu. yd.	at $0.75	$16.65	$16.65
Quarrying stone	23 " "	" 0.50	11.50	
Quarrying stone	15 " "	" 0.50		7.50
Hauling field stone	15 " "	" 0.25 (in place).		3.75
Crushing quarry stone	23 " "	" 0.25	5.75	
Crushing quarry stone	16 " "	" 0.25		4.00
Hauling crushed stone	23 " "	" 0.32	7.36	
Hauling crushed stone	16 " "	" 0.32		5.12
Placing and rolling	133.3 sq. yd.	" 0.15	20.00	
Placing and rolling	133.3 " "	" 0.10		13.33
Comparative totals			$61.26	$50.35

These figures are not intended to fit any particular case, but are taken as an average from cost items on several roads. The first item

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will vary considerably, and will affect somewhat the total percentage saved under the proposed method. The cost of hauling field stone may vary, also, but, if stone walls are convenient, and if the stones are of moderate size or easily broken, the figure used may be reduced. The quantities used are, 23 cu. yd. of stone for the present method and 30 cu. yd. for the proposed method, giving a thickness of finished macadam of 6 in. in the former case and 8 in. in the latter. In the proposed method it is estimated that 50% of the volume will be crushed stone.

Where there are no field stones, but a good quarry is at hand, the estimate would be changed, as follows:

				Present method.	Proposed method.
Preparation of base				$16.65	$16.65
Quarrying stone	23	cu. yd. at	$0.50	11.50	
Quarrying stone	30	" " "	0.50		15.00
Crushing stone	23	" " "	0.25	5.75	
Crushing stone	16	" " "	0.25		4.00
Hauling crushed stone	23	" " "	0.32	7.36	
Hauling crushed stone	16	" " "	0.32		5.12
Hauling quarry stone	15	" " "	0.32		4.80
Placing and rolling	133.3	sq. yd. "	0.15	20.00	
Placing and rolling	133.3	" " "	0.10		13.33
Comparative totals				$61.26	$58.90

Under such conditions as abundant field stone with no quarry suitable for top stone, so that top stone would be imported in any case, trap might be specified for the top, and the estimate would be about as follows:

				Present method.	Proposed method.
Preparation of base				$16.65	$16.65
Hauling field stone	16	cu. yd. at	$0.25	4.00	
Hauling field stone	22	" " "	0.25		5.50
Crushing field stone	16	" " "	0.25	4.00	
Crushing field stone	11	" " "	0.25		2.75
Hauling crushed stone	16	" " "	0.32	5.12	
Hauling crushed stone	11	" " "	0.32		2.52
Trap, dumped in place	8	" " "	3.50	28.00	28.00
Screenings (limestone)	5	" " "	2.25	12.25	12.25
Placing and rolling	133.3	sq. yd. "	0.15	20.00	
Placing and rolling	133.3	" " "	0.10		13.33
Comparative totals				$90.02	$81.10

Although these figures cannot be exactly determinate, owing to inability to apply them, as they are, to any particular road, the writer

Mr. Wedgeworth. believes that, as an average, they are fair, and that they indicate a reduction in cost and the production of a superior roadway, in favor of the proposed method.

It may be said, in opposition to this method, that it is impossible to roll the loose stone of such depth into the spaces among the large stones of the base. This, however, the writer does not believe to be the case, as the sub-base is being constructed by this method on many roads. Why not make the roadway itself a sub-base?

One often hears it said that macadam roads cost too much, both at first and in maintenance, yet contractors are slow to take them at appropriation figures. It is the duty of the engineer to furnish the best construction for the least money possible. The effort at improvement, however, has been directed toward making the present method more thorough and more expensive.

Macadam is being laid over comparatively good roads, built before the days of rollers and stone crushers. In the finer product let engineers not forget entirely the methods and means of those who built before them, for they built well and at little cost. More stone and less expensive manipulation is a step in the right direction. Try it.

Mr. Baker IRA O. BAKER, M. AM. SOC. C. E.—In many of the States of the Mississippi Valley there are large tracts of rich alluvial soil almost entirely devoid of either gravel or stone suitable for building wagon roads. Many of these tracts are at present in a high state of cultivation, and all of them promise to be comparatively soon. Some of these tracts are probably as rich and productive as any equal area in the world; and the increasing population of the country and the intensity of the scientific study now being bestowed on agricultural operations in the United States promise speedily to increase still further the productiveness of the soil and thus add to the wealth of these communities.

The problem of securing and maintaining suitable wagon roads in these districts is not a simple one, either in its political or its material aspects. There is great diversity of opinion as to what constitutes a road suitable for the needs of these communities, some contending that the earth road meets all the necessary requirements, and others that the gravel or macadam road is an economic and social necessity. It is the purpose of this discussion to call attention to some of the difficulties in the case, although the questions involved are political, social, and economical, rather than engineering.

Before considering the subject positively, it may be wise to look at it negatively; that is to say, before trying to discuss the problem one should get clearly in mind what it is not. During the 10-year period beginning about 20 years ago a large amount of literature upon the subject of rural road improvement was set afloat in newspapers, magazines, and reports, and, because of this flood, the wagon roads

of the country districts have made less progress in the last 15 or 20 years than any other phase of rural life. Mr. Baker.

Facts and Theories Inapplicable.—About 20 years ago, two or three well-organized manufacturing interests, in order to increase the sale of their goods, deliberately started an agitation for better roads, and, unfortunately, committed the conduct of the scheme to those who knew little or nothing about either road construction or the larger principles of economics and political action involved in any comprehensive system of road improvement. The interests back of this propaganda were very skilful in arousing the enthusiasm of their followers, who were both numerous and widely distributed, and an attempt was made to carry by storm a reform system of constructing and maintaining the wagon roads of the nation. Many politicians and newspapers sought to secure popularity by favoring the proposed reformation, and, as a consequence, wide circulation was given to certain facts and theories, which in their proper field were true, but which when presented without limitation were wholly false and entirely misleading.

These intemperate advocates of road reform frequently cited the wagon roads of England, France, and Switzerland as examples of what the United States could and should do, regardless of the fact that those countries are abundantly supplied with good road-building materials, while in this country there are areas, greater than the combined area of those countries, in which there is absolutely no road-building material. Again, the stone wagon roads in those countries were built under the stimulus of military necessity and commercial need, before the advent of steam railroads; while in the greater part of this country the railroads have been the pioneers, and now there is no commercial need of long lines of wagon transportation, except perhaps in the immediate vicinity of large cities. Further, the density of population, the industrial occupation, and the agricultural methods in those countries are very different from the methods prevailing in this. Still again, the method of maintaining wagon roads in those countries, with their dense and poorly-paid population, is no criterion by which to judge what is wise or possible in this country. And once more, the political and social ideals on the two sides of the Atlantic are very different, and make possible certain results in Europe which are impossible in the United States.

Again, these enthusiasts frequently cite the experiences of local cities on Long Island and in New Jersey, within a few miles of New York City, as though they were representative of rural conditions in general. New York is the largest and wealthiest city on the North American continent, and it is not to be wondered at that the building of good roads is one means of enticing people to move from the densely populated districts of the city into the more healthful suburban regions, and that the influx of such inhabitants has

Mr. Baker. greatly increased the value of real estate; but not all of the United States is situated in the bedroom of New York City. Some years ago the annual report of the State Highway Commissioners of New Jersey contained an interesting account of road improvement in Gloucester County, which has been very widely copied and commended; but when it is known that this county is just across the river from Philadelphia, and that the soil is very sandy and the road improvement simply enabled truck farmers to wagon their produce to market, it is seen that this example has but little practical bearing on the general question of rural road improvement. Again, along the eastern shore of New Jersey there is a continuous line of summer resorts, where people of wealth go to enjoy themselves, and it is doubtless a good business proposition for such communities to build first-class stone roads for the pleasure of their profitable guests; but their success does not prove that a truly rural community should follow their example and also build expensive roads. The examples of the States of Massachusetts and New York in hard-road building are frequently commended to the inhabitants of the Prairie States of the Mississippi Valley, but the industrial and topographic conditions of these two States are not guides for the flat, grain-raising States. The primary fault of most road reformers has been that, through dense ignorance, superficial knowledge, or deliberate intention, they have presented facts, separate and apart from their limitive conditions, in such a way that the truth becomes a falsehood.

This ill-advised and intemperate road agitation has in many cases done harm in the communities most needing road improvement, since a form of road construction has been advocated which, under most conditions, has been entirely impracticable, if not ridiculous; and the result has been that those who are most interested in good roads have been most concerned to prevent road construction and maintenance being made subservient to interests foreign to those who use the roads most and also pay for them; and, as a result of this condition of affairs, the improvement of local roads has not received the attention recently that its importance warrants, and has not kept pace during the past 20 years with rural development in other lines.

Furthermore, all these road enthusiasts claimed that the only good road was a hard road; they were also very vehement in the assertion that nothing could be done to improve the earth road. In many localities a hard road was politically and economically impossible, and hence the earth road was the only form available; but the assertions of the good-road enthusiasts only serve to fix the belief that nothing could be done to improve the roads of such localities.

Fallacies in Road Economics.—It was in the field of economics that these road reformers made their most glaring errors, produced the most irritation, and aroused the most antagonism. In this field the farmer

was most capable of independent judgment. He was not much concerned about the lessons which the road enthusiast sought to draw from European experience or even from experience along the Atlantic Coast, for the farmer knew that social and industrial conditions in those localities were very different from those in the Mississippi Valley, and hence such arguments made little or no impression on him; but when the hard-road enthusiast began to tell the farmer how much it cost him to haul his produce to market, and how much he could save by the construction of hard roads, he knew instinctively that the conclusions were ridiculous, and the continual harping upon these false statistics and absurd estimates led him to believe that an attempt was being made to force hard roads upon him, whether or no, and his attitude changed from one of indifference to one of open hostility to all road improvement. Mr. Baker.

A brief examination of a few of the claims of these agitators will be made in order to see whether or not the farmer was justified in his judgment as to their truthfulness. It would not be worth while to refer to this literature except that by so doing its nature may be exposed and thus prevent it from being used in similar discussions in the future. The speaker will examine three articles—of the most reputable parentage—which have been most widely circulated in their original form, and are most frequently quoted. These articles seem to have been distributed to all the public libraries, and, apparently, when any one is moved to prepare a speech on road improvement or write an article on that subject to be used as plate matter in the country newspapers, he consults such literature. The speaker has seen evidence that each of the articles about to be referred to was used recently in connection with the meetings called to secure a good-roads plank in the platforms of the two principal political parties.

First.—One of these articles is a 64-page pamphlet entitled: "The Gospel of Good Roads, A Letter to the American Farmer," published by The League of American Wheelmen. The author says:

"From official Government sources I find that the farmers of this country, in the year 1890, and upon their farms draft animals as follows:

"Kind.	"Number.	"Value.
Horses	14 213 837	$978 516 562
Mules	2 331 027	182 394 099
Oxen	36 849 024	560 625 137
Total	53 393 888	$1 721 535 798

"You see you have nearly $2 000 000 000 invested in motive power of a perishable, uncertain, and expensive kind. Busy or idle, these animals must be fed and cared for every day. They are boarders that you can't get rid of when the busy season is over, and it stands you in hand to keep them at work."

Mr. Baker The author of the above had surely not traveled much, or he would have learned that there were not anything like two and one-quarter times as many oxen in this country as horses and mules. Evidently, where the census report said cattle, he read oxen! The man refers to a fellow hard-road enthusiast who "estimates" that "bad roads cost the farmer $15 per year for each horse and mule;" and calculates that this loss amounts to $250 000 000 per annum. He next asserts, apparently offhand, that the loss due to "wear and tear on wagons and harness is $100 000 000." He then proceeds to add together the value of his draft animals, the alleged cost of bad roads, and the estimated cost of wear and tear on wagons and harness, and gets a total of $2 350 000 000. The sum is frequently quoted as the annual cost of bad roads in this country.

The above computations are almost as good as anything in *Puck.* Were not some of the horses and mules utilized in the cities? Think of the ludicrousness of including the value of each horse and mule in the cities (including those then used on the street railroads) plus $15 in determining the cost to the farmer of bad roads. Isn't it cruelty to animals to make the colt in the farmer's pasture bear the burden of a draft animal; or to charge the maverick on the plains with being responsible for part of the cost of bad roads? The wear and tear on wagons and harness had already been included in the preceding $250 000 000, the cost of bad roads; but an extra $100 000 000 is nothing to a hard-road enthusiast. Of course it is a small matter to have added the value of the draft animal to the alleged difference of its earning power on good roads over that on bad roads! The article makes an error of $2 100 000 in determining $250 000 or less! But such an error is of no moment to the road reformer!

There are several other things in this pamphlet almost as misleading, but the foregoing item is referred to here because it is the part most frequently quoted.

The pamphlet is liberally illustrated by cuts made from prize photographs collected by the members of the League, showing the impassable condition of earth roads, and also by cuts of some of the magnificent roads of Europe. As well show a picture of some of the hovels on the rocks on the north end of Manhattan Island or of a 46-story building, as representative buildings of New York City.

Second.—The article which has been quoted most frequently is one published by the United States Government, in which it is distinctly stated that the cost of wagon transportation in 1895 was $946 314 665.54 and that the possible annual saving by road improvement is $628 000 000. This article also distinctly asserts that the cost of hauling the farm crops to market is $662 000 000, or 26.6% of their value.

This report has been criticized in detail elsewhere, and hence needs no further examination here. With a certain class this publication

had great influence, as it appeared to have the weight of the United States Government, and as it claimed to be the result obtained in answer to 10 000 letters sent to agricultural correspondents all over the country; but farmers of the rich grain-producing areas of the Mississippi Valley, the districts most in need of road improvement, protested loudly against any such estimate. They claimed that, through the exchange of work and by doing their hauling when other work was not pressing, the marketing of their crops cost them practically nothing. Mr. Baker.

Ten years after the above report was published, the United States Government published the results of another and more elaborate investigation, in which it is distinctly stated that the cost of hauling the 1905 crops to market was $84 684 000, or only one-eighth of the result in the former report! Other Government statistics show that the crops of 1905, the year of the last report, were practically twice as large as those of 1895, the year of the first report; and therefore the result by the later and more elaborate investigation is really only one-sixteenth of that of the first report! In other words, the result by the first report is virtually admitted to have been sixteen times too large! Is it any wonder that the farmer was unwilling to accept such a result?

As illustrating the fatality that seems to overtake hard-road enthusiasts when they attempt to compute the cost of bad roads, or rather as illustrating their proneness to look for excuses for making their results larger, the fact may be mentioned that in the latter investigation the author added $11 700 000 for the wheat that was hauled from the farm to the mill, which by his own figures is equivalent to saying that nearly one-third of all the wheat raised in this country is hauled in wagons from the farm to the mill. No statistics are at hand to check this item, but the probability is that the amount of wheat hauled in wagons to the mill is a very small proportion of the total; and hence this correction is much too large.

There are three other errors in the above investigation that make the conclusion too large: 1. The correspondent was asked: "What is the greatest distance of haul to shipping point by any considerable number of farmers?" The answer was assumed to be the radius of the contributing area, regardless of the fact that usually, because of topographic or other conditions, produce is hauled much farther from one direction than from another. This makes the distance of haul too great. 2. The average weight of load was assumed to be the mean between the largest and the smallest, regardless of the fact that most of the farm products go to market when the earth roads are in nearly their best condition, and consequently the maximum load is very much more common than the minimum and not equally frequent as was assumed. This error makes the assumed average load too small,

Mr. Baker. and therefore makes the computed cost of marketing farm products too great. 3. The price per day of team and driver was taken as the "usual cost of hiring a team and driver," regardless of the fact that hauling is a secondary employment to farmers and that the conditions of service, cost of feed and driver, loss of time by bad weather, etc., are very different for the farmer and the professional teamster. The farmer claims that the assumed price per day for team and driver is much too great.

The above total, \$84 684 000, is probably at least twice too large. However, even this sum is only \$15 each for the 5 740 000 farmers of the United States; and therefore the alleged cost of bad roads in marketing the crops is not likely to bankrupt the farmer. Of course, only part of this sum would be saved if the farmer had permanently hard roads upon which to haul; and consequently only a fractional part of this sum is available for hard-road construction, if only the economics of the problem is considered. Good roads are of an advantage to any rural community, but they must be defended chiefly for other than economic reasons.

Third.—The third article referred to is a speech by a United States Senator in the Senate in 1904, which was circulated as a public document. The Senator also traveled about over the country, apparently making the same speech. The speaker heard it twice in one day in the same room. The Senator said that an Illinois farmer owning 100 acres can get permanently hard roads for a tax of \$20 per year for 5 years, provided the State or National Government will pay an equal amount; and then he will have three ways in which he can make 100% on his investment. The speaker will examine this statement briefly.

Under the most favorable assumptions, the above tax amounts to \$1 280 per mile. What kind of a permanently hard road can be built anywhere for that sum? What kind of a road can be built for this sum in the Illinois corn belt, where the Senator was speaking and where gravel or broken stone must be hauled about 100 miles?

The Senator's first method of making 100% on the road tax is as follows:

"The farmer would get at least 52 days' labor, when not engaged in his crop, with his team at \$2.50 per day which would amount to \$130, of which half, or \$65, would be clear profit."

In the first place, the tax is \$100, but the Senator only claims \$65 profit when he promised \$100! In the second place, as the farmer and the Government together only pay \$200, can 65% of it be safely spent for hauling the material?

The Senator's second way of paying the road tax was by the reduced cost of hauling produce to market. In computing the cost on earth roads he virtually counted the cost of team and driver at \$3.75 per day, even though he had just said that at \$2.50 per day half was

clear profit! He also counted the produce per acre more than four times that given by the U. S. Census. Granting his estimates and assumptions, he showed a profit under this head. Mr. Baker.

The Senator's third way of paying the road tax was by the increase in the value of the land. All hard-road advocates count both the reduced cost of transportation and the increase in the value of the land, regardless of the fact that the second is the result of the first, and also regardless of the further fact that hard roads add nothing to the productivity of the soil.

The Effect of the Good-Road Agitation.—Attention has been called to some of the absurdities of three of the most prominent pieces of good-road literature known to the speaker; and, if it were desirable, he could present others which are equally startling, although not of as good parentage. When some callow bicyclist writes on road reform for his local paper, or some automobile agent writes on roads for a trade journal, or a carriage dealer makes a speech at an annual convention, the statements of the three articles referred to are accepted because of their authorship, and the changes are rung upon their statements without any question as to their truthfulness. Accepting the above statistics and estimates as true, the conclusion is drawn that the farmer is a fool not to act thereon, and consequently such productions not infrequently bristle with opprobrious terms applied to the farmer.

Is it any wonder that the farmer has not been influenced, at least favorably, by such literature? What would be the effect if the dairymen, the corn growers and the cattle raisers should flood the cities with literature calling attention to the imperfections of street pavements, and claiming that the metropolitan residents were losing each year vast sums of money through lack of interest in the conditions of the pavements, and should cite noted boulevards and park drives as the kind of pavement the cities could and should have on all their streets?

Many of the advocates of hard-road construction have not had an adequate comprehension of the facts and figures they have presented, and have greatly underrated the understanding of the farmers they wished to convert. The men who make public opinion in any rural community know that, while good roads may have greatly enhanced the value of real estate in the bedroom of some great city, such conditions cannot be widespread. The representative farmer understands the difference between the conditions under which he labors and those of the huckster near a large city. Even though hard roads may enable a farmer now and then to rush to town with a dozen of eggs, or a bushel of potatoes, or perhaps a load of hay, and obtain a fabulous price therefor, he knows that these conditions are exceptional; and also that, if any considerable number of hard roads are built, producers must accept the general level of prices. The

Mr. Baker. Illinois farmer understands the difference between his State and Indiana in the matter of the proximity of road-building material; and he also understands the difference between the rich, sticky soil of his own State and the soil of Massachusetts, and believes that the experience in the Bay State is not a trustworthy guide for him. It may add spice to the article to embellish it with pictures showing wagons literally half buried in mud; but any man with sense knows that these conditions are not representative.

Within the past few years the agitation for hard roads has nearly ceased in the Prairie States, and, coincident with such subsidence, increased attention has been given to earth roads. When the hard-road agitation was actively going on, there was little or no demand in the farmers' institutes (meetings of the farmers to discuss agricultural topics) for articles or discussions on road topics; but recently there has been a large demand for information concerning the care of earth roads. This is very fortunate, for almost, if not absolutely, universally in this country the administration of road affairs is in the hands of small local official boards, which from the nature of our form of government are likely to change frequently. This condition imposes a well-nigh insurmountable limitation upon any comprehensive and continuous system of road improvement, unless the general public firmly believes in the value of the proposed system. Therefore it is highly important that correct information concerning road economics, road administration, and road construction should be widely disseminated. Unless a community is willing and able to maintain the earth roads in a reasonably good condition, it is useless to expect that it will be willing or able to support a high-class wagon road; and therefore the dissemination of correct information concerning the construction and care of earth roads is politically, economically, and physically the first step toward a better form of construction.

The problem of the earth road will now be considered briefly.

Drainage of Earth Roads.—Drainage is the most important matter to be considered in the construction of earth roads, since no road, whether of earth or stone, can long remain good without it. Water is the natural enemy of earth roads, for, mixed with dirt, it makes mud, and mud makes bad going. The rain or snow softens the earth; the horses' feet and the wagon wheels mix and knead it; and soon the road becomes impassable mud, which the frost finally freezes, the second state of the road being worse than the first—for a time at least. Further, if the water is allowed to course down the middle of the road, it will wash away the earth, and leave gullies in the surface that must be laboriously filled up by traffic or by repairs. Prompt and thorough drainage is a vital essential in all road construction, and particularly so for earth roads.

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For a road on loam or clay there are three systems of drainage, each of which must receive attention if the best results are to be obtained. These three systems are: (1) underdrainage, (2) side ditches, and (3) surface drainage.

Underdrainage.—Any soil in which the standing water in the ground comes at any season of the year within 4 or 5 ft. of the surface will be benefited by drainage; that is, if the soil does not have a natural underdrainage, it will be improved for road purposes by artificial subsurface drainage. It is the universal observation that roads in low places which are underdrained dry out sooner than undrained roads on higher land. Underdrained roads never get as bad as do those not so drained.

There are three distinct advantages of underdraining earth wagon roads.

The most important object of underdrainage is to lower the water level in the soil. The action of the sun and the breeze will finally dry the surface of the road; but if the foundation is soft and spongy, the wheels will wear ruts and the horses' feet will make depressions between the ruts. The first shower fills these depressions with water, and the road is soon a mass of mud. A good road cannot be maintained without a good foundation. An undrained soil is a poor foundation, while a dry subsoil can support almost any load.

A second object of underdrainage is to dry the ground quickly after a freeze. When the frost comes out of the ground in the spring, the thawing is quite as much from the bottom as from the top. If the land is underdrained, the water when released by thawing from below will be immediately carried away. This is particularly important in road drainage, since the foundation will then remain solid and the road itself will not be cut up. Underdrainage will usually prevent the "bottom dropping out" when the frost goes out of the ground.

A third, and sometimes a very important, object of subdrainage is to remove what may be called the underflow. In some places where the ground is comparatively dry when it freezes in the fall, it will be very wet in the spring when the frost comes out—surprisingly so, considering the dryness before freezing. The explanation is that, after the ground freezes, water rises slowly in the soil by the hydrostatic pressure of the water in higher places; and, if it is not drawn off by underdrainage, it saturates the subsoil and rises as the frost goes out, so that the ground which was comparatively dry when it froze is practically saturated when it thaws.

The underdrainage of a road, not only removes the water, but prevents, or greatly reduces, the destructive effect of frost. The injurious effect of frost is caused entirely by the presence of water, and the more water there is in the roadbed the greater the injury to the road. The

Mr. Baker. water expands on freezing, the surface is upheaved, and the soil is made porous; when thawing takes place, the ground is left honeycombed and spongy, ready to settle and sink, and under traffic the road "breaks up." If the road is kept dry, it will not break up. Underdrainage cannot prevent the surface of the road from becoming saturated with water during a rain; but it is the best means of removing the surplus water, thus drying the surface and preventing the subsequent heaving by frost.

Side Ditches.—The side ditches are to receive the water from the surface of the traveled way, and should carry it rapidly and entirely away from the roadside. They are useful, also, to intercept and carry off water that would otherwise flow from the side hills upon the road. Ordinarily, they need not be deep, but, if possible, should have a broad, flaring side toward the traveled way, to prevent accident if a vehicle should be crowded to the extreme side of the roadway. The outside bank should be flat enough to prevent caving. If the road is tiled, the side ditch need not be very large; but it should be of such a form as to permit its construction with the road machine or scraping grader, instead of by hand. On comparatively level ground, the proper form of side ditch is readily and cheaply made with the usual road machine or scraping grader. A deep, narrow ditch is expensive to maintain, since it is easily obstructed by the caving banks, by weeds, and by floating trash. If it is necessary to carry water along the side of the road through a rise in the ground, it is much better to lay a line of tile and nearly fill the ditch than to attempt to maintain a narrow, deep ditch.

The side ditch should have a uniform grade and a free outlet into some stream, so as to carry the water entirely away from the road. No good road can be obtained with side ditches that hold the water until it evaporates. Most ostensible road work is a positive damage, for this reason. Piling up the earth in the middle of the road is perhaps in itself well enough; but leaving undrained holes at the side probably more than counterbalances the benefits of the embankment. A road between long artificial ponds is always inferior, and is often impassable. It is cheaper and better to make a lower embankment, and to drain thoroughly the holes at the side of the road.

Surface Drainage.—The drainage of the surface of a road is very important, and is provided for by making the surface crowning and keeping it smooth. It should be remembered that water upon the surface of the road cannot be carried away by the underdrains, since the water can reach them only after it has penetrated and softened the road surface. On account of the puddling of the surface when wet, by the action of the hoofs and wheels, and, under the most favorable conditions, little water will percolate through the surface and reach the tile, and, with clayey soil, no water will be thus removed, the slope

from the center to the side should be enough to carry the water freely and quickly to the side ditch; and, if the surface is kept free from ruts and holes, less crown will suffice than if no attention is given to keeping the surface smooth. If there is not enough crown, the water cannot easily reach the side ditches; and hence the road soon becomes water-soaked. Surface drainage is chiefly a matter of maintenance, and will be referred to again under that head. Mr. Baker.

Maintenance of Earth Roads.—The most important work in maintaining an earth road is to keep the surface smooth so that the rainwater will flow quickly into the side ditches. If the surface of the roadway is properly formed and kept smooth, the water will be shed into the side ditches and do comparatively little harm; but if it remains upon the surface, it will be absorbed and will convert the road into mud. If all ruts, depressions, and mud holes are not filled as soon as they appear, they will retain the water upon the surface, to be removed only by gradually soaking into the roadbed and by slowly evaporating; and each passing wheel or hoof will help to destroy the road.

There are several machines or devices which are very effective in filling ruts and depressions, and in keeping the surface smooth. Among these are the ordinary farm harrow, a square stick of timber shod on one edge with a strip of steel, a railroad rail, three or four 2-in. planks with their edges lapped and nailed, the **A**-drag, which consists of two planks set on edge vertically to form a letter **A**, and the split-log drag.

The road machine, or scraping grader, or road plane, a cutting blade suspended obliquely under a frame resting upon four wheels, is much used to smooth up the road in the spring; but it is not as good for this purpose as the more simple devices mentioned above. It is heavy, and cannot be used until the roads are too dry and hard for the most efficient work, and requires four, and usually six, horses and two men to operate it; while the other devices require only two horses and one man, and, if used at the proper time, are more effective.

The harrow is an efficient instrument in leveling the road just as the frost is going out, and also in smoothing the road in the summer when the surface has become rough.

All the other devices or machines act upon the same principle, that of filling up the ruts and depressions and working a little earth toward the center of the road to maintain the crown of the road by counteracting the wash of the rains. The timber or the railroad rail or the plank drag is drawn along the road with its length nearly perpendicular to the traveled way, but with the end toward the center of the road a little behind the other. By changing the angle of the cutting edge with the line of draft, according to the

Mr. Baker. condition of the road, enough earth is pushed along in front to fill the ruts and depressions and also to work a little toward the center of the road to maintain the crown. The **A**-drag is drawn over the road with the pointed end forward. Some prefer this form of drag to the split-log drag; but there is not much difference either way, and, as the latter is more frequently used, it alone will be described in detail here.

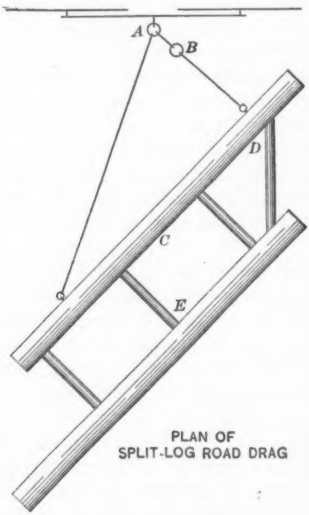

FIG. 3.

Farmers in different parts of the country, for many years, have used some of these devices occasionally in smoothing the surface of the earth roads; but, of all these, none seems to have devised a better form of machine or been more persistent and intelligent in its use, or to have been more successful in interesting others in its use, than Mr. D. Ward King, of Maitland, Mo. Mr. King devised what he calls the split-log drag. A plan of the split-log drag is shown in Fig. 3, and Fig. 4 is a perspective view. The drag may be made from a log 10 or 12 in. in diameter and from 7 to 9 ft. long. A light wood, like elm, is preferable to a heavy one, like oak. The cross-braces may be round or square sticks from 3 to 4 in. in diameter, the ends fitting into 2-in. auger holes. A board, not shown in the cut, is laid upon the cross-pieces for the driver to stand upon. The drag may also be made of two pieces of plank 10 or 12 in. wide and

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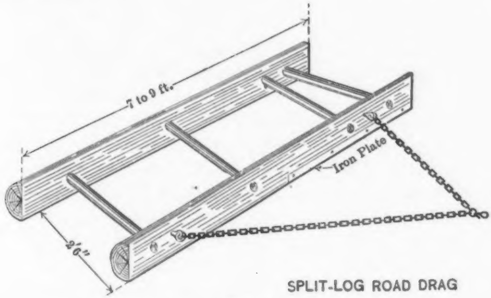
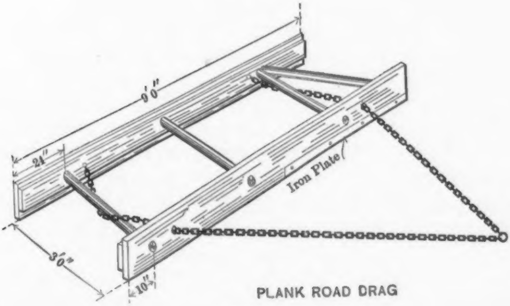
from 7 to 9 ft. long. The plank drag is shown in Fig. 5. It is wise to reinforce the wide planks with either a 1 by 6-in. or a 2 by 6-in. strip, as shown in Fig. 5. In Fig. 5 the chain is shown as being fastened on the rear side of the rear plank, which insures that the front plank will not be pulled off the cross-pieces—not a very important precaution.



SPLIT-LOG ROAD DRAG

FIG. 4.



PLANK ROAD DRAG

FIG. 5.

The drag is drawn by two horses, and its length should be proportional to the weight of the horses. A drag 7 ft. long is about right for a team of 1 200-lb. horses, and one 9 ft. long for two 1 600-lb. horses. The driver rides upon the drag, and varies its effect by his position upon it. The drag does the best work when the soil is moist, but not sticky. If the roadway is badly rutted and full of holes, it is well to drag it when the surface is slushy.

The use of the split-log drag improves the common earth road in three important ways: First, it smooths the road, which facilitates the drainage of the surface and also gives a better surface to drive upon. Second, the use of the drag moves earth toward the center of

Mr. Baker. the road, thereby increasing or at least maintaining the crown of the road, which is necessary for good surface drainage. Third, if the drag is used when the surface is wet, the earth will be puddled, and then, when the sun shines, the earth will be baked—both of which will aid materially in making the road hard. Another valuable result of dragging the road when it is wet is the reduction of dust, for the particles of clay cohere more tenaciously and less dust is produced.

The cost of maintaining earth roads varies with the nature of the soil, with the weather, and with the degree of excellence attained; but, in a number of localities, roads on clay and on loam are greatly improved at an expense of from \$3 to \$5 per mile per annum, allowing from 30 to 40 cents per hour for driver and team. The work can be done to the best advantage by farmers who drag the road adjoining or near their own land, as it frequently happens that there is only a comparatively short time when the soil is in suitable condition for dragging. Fortunately, the best time to drag the roads is when it is too wet to work in the field.

Labor vs. Money Tax.—Road reformers usually grow eloquent in denouncing the labor tax system of caring for the public highways, and generally claim that the common earth roads are poor only because of the method of working out the tax. A little study of the facts and conditions will show that such claims are wide of the mark.

The labor-tax system is regularly used in all the States of the Union except five. In one State at least (Illinois) the road tax may be collected in money or labor as the township by election may decide, and a large majority of the towns vote in favor of the labor system. The labor-tax system was inherited from England, and is a survival of the feudal method of requiring all able-bodied men to render public service. England and France have a labor road-tax, but upon a much less extensive scale than the United States.

It is common to assume that the labor-tax system is all wrong, and that its evils would be escaped by paying road taxes in money. The labor tax has inherent disadvantages, but many of the defects charged to it belong rather to defective administration and to the system that leaves the control of the public highways to a small locally-governed community. Public work is seldom done as economically as private work.

The objections to the labor-tax system are: 1. The labor is indifferent and inefficient. 2. It is impossible to get the work done at the most suitable time. 3. The system allows no selection of the laborer. All of these are important considerations.

The reply to the above objections is usually about as follows: 1. The farmer is willing to pay more in labor than in money, which compensates in part, at least, for the objections to the labor-tax

system. This preference is not peculiar to the American farmer. In France, if the road tax is paid in money, a reduction of from 40 to 50% is made; but still 60% of the people prefer to pay in labor. Farmers not infrequently give more, both in labor and money, than is exacted as road taxes, because they are interested in better roads. 2. In many rural communities it is impossible to secure anyone to do road work at reasonable wages at the most suitable season. 3. If the tax were paid in money, there is no certainty that the labor would be any more efficient. Streets are maintained under the cash-tax system, but the labor is not ideally efficient. The authority that virtually wastes the labor tax will probably also waste the cash tax. Mr. Baker.

The labor tax is not necessarily the cause of inferior roads, nor the cash-tax system in itself the cause of improved roads. Townships under the labor-tax system often have better roads than adjoining townships under the cash-tax system. The one thing absolutely necessary for successful road management is effective supervision of the work. Without it, neither system will accomplish much, and, with it, either system will do reasonably well.

Many townships have changed from the labor-tax system to the cash-tax system with a marked improvement in the condition of the roads—due chiefly, if not wholly, to better administration. For in many of these cases the so-called cash-tax system is practically only a change in the method of administering the labor-tax system, since farmers desiring to do so are given an opportunity to work out their road taxes under the cash system. Under the labor-tax system those working upon the roads receive credit on their road taxes, while in the so-called cash system the laborer receives an order which is accepted as cash in paying taxes. In these cases the public sentiment that demanded road improvement secured the change from the labor tax to the cash tax; and, consciously or unconsciously, also secured a more efficient road administration.

Maintenance by Contract.—In view of the ordinarily inefficient system of caring for roads, it has frequently been proposed to maintain them by contract. As a rule, work done under the supervision of a contractor who has pecuniary interest in the result is more economical than that performed under the direction of a public official; but it is not wise to do work by contract unless the amount required can be approximately known beforehand, and also unless the character of the performance can be easily determined after completion. Neither of these important conditions would be present in a contract for the maintenance of a public highway. Owing to the indefiniteness as to the amount and character of the work to be done, it is not at all certain that the maintenance could be provided for by contract for a sum less than the public officials could do the work under the present system. The inspection would finally depend on the road official, and the

Mr. Baker. letting of a contract would increase the difficulties and expense of supervision.

Under the present system, those who perform the road labor have an interest in the resulting condition of the roads, while the contractor would be interested only in doing the work for the least money; and therefore the roads would probably be worse under the contract system than under the present system.

It is claimed that the contractor could maintain a trained corps, and therefore do better work than can be obtained by the present system. This would possibly be true if the amount of work to be done were sufficiently great; but statistics show that the amount expended upon earth roads is only $40 or $50 per annum, and a large part of this is for bridges, which are built by contract, and a considerable part of the remainder is for tile and lumber for bridge floors, culverts, etc., while an additional sum is paid for laying tile and for dredged outlets, both of which usually represent contract work. Thus the amount remaining to be spent for the care of the traveled way and of the roadside is quite small; and therefore the ordinary expenditure for the care of earth roads is too small to justify maintaining a corps of expert road workmen. Further, leaving the road work to a comparatively few trained attendants would result in a great waste of time in traveling to and from the work. Again, the attendant would have so many miles of road under his care that he could visit any particular piece only at long intervals, and therefore could not do the work at the most favorable time, and could not become intimately acquainted with the road—conditions absolutely necessary for proper maintenance. These objections have less force as road expenditures increase, and as the money is concentrated on a comparatively few roads. Finally, a large proportion of the roads have an earth or gravel surface, and the labor required for their care is similar to that with which the farmer is familiar; and therefore he is not lacking in the skill required in maintaining them. The farmer who travels a particular road frequently and in all kinds of weather has a more intimate knowledge of it than the man who sees it only occasionally; and therefore, for this reason, the farmer is best able to care for the road. Besides, the farmer uses the road more than anybody else, and he alone pays for it.

The system of employing a man to give his entire time to the road is almost a necessity with first-class broken-stone roads, the maintenance of which requires intimate knowledge and constant attention, but the system is not applicable to earth roads.

Real Difficulties.—One of the most serious difficulties in the way of better earth roads is that the administration of road affairs is in the control of officers who are changed frequently and who devote only a small part of their time to the roads; and it is not clear that this diffi-

culty can be overcome. It has been proposed to commit the general oversight of the roads to a county superintendent, much as the schools now are; but this plan is not very promising. The county superintendent of schools can hold examinations to determine the fitness of teachers; but a county superintendent of roads could not use such a method to control the efficiency of road workers. The county superintendent of schools can visit each teacher while at work, but a county superintendent of roads could hardly expect to do correspondingly for the road workers. A county superintendent of roads could render valuable public service in inspecting highway bridges and in preparing plans and letting contracts for new ones; and possibly might do something by holding public meetings for instruction in and discussion of road work, but it is not sure that the farmers could or would attend, and besides this function is already performed by the usual farmer's institute. Mr. Baker.

Another difficulty in the way of any great improvement of the common roads is that much of the richest agricultural land—that which needs better roads most and which is most able to pay for them—is held by non-resident owners; and the men who have opened farms and brought them to a high degree of cultivation are leaving them and moving to the nearby small town or more remote city. The non-resident owner is not usually interested in the improvement of general conditions that do not bring him immediate financial returns; and the removal of the land owner lowers the general intellectual average of the rural community.

With the increasing introduction of agricultural machinery, the farms have been growing larger and larger and the number of residents in the country has been growing smaller and smaller, and consequently there are fewer people to be benefited by improved roads.

Signs of Promise for the Future.—Within the last few years the extravagant advocacy of hard roads has nearly ceased, and, almost coincident therewith, attention has been directed to the improvement of the earth road. This subject is considered at nearly every farmer's institute, and is treated in editorials and correspondence in the agricultural newspapers. Several of the States have State Highway Commissions which are devoting much time and money to experiments, to public addresses, and to the preparation and circulation of literature on the earth road.

Good roads add to the social, educational, and intellectual welfare of the rural community, facilitate freedom of intercourse between the dwellers on the farms and in the cities, and thereby contribute to the progress and stability of the country. Therefore, all should be willing to do all they can to help along the improvement in this line.

Arthur H. Blanchard, Assoc. M. Am. Soc. C. E. (by letter).—It was the original intention of the writer to present, as a closing discussion, a Mr. Blanchard.

Mr. Blanchard. review of American and European practice covering the maintenance of macadam roads subjected to excessive motor-car traffic. Since the Denver Convention, however, two reviews have been published in book form: one by William P. Judson, M. Am. Soc. C. E., entitled "Road Preservation and Dust Prevention," and the other by Prevost Hubbard, Assistant Chemist, United States Office of Public Roads, entitled "Dust Preventives." The writer, therefore, will conclude with a reply to the query of Mr. Nelson P. Lewis relative to the maintenance of bituminous macadam, and a description of the bituminous macadam work completed on the East Providence contract since June, 1908.

Although it has not been necessary to repair any of the sections of bituminous macadam constructed in Rhode Island, the writer believes that satisfactory repairs can be made by filling any holes which may occur with tarred stone of a size depending upon the dimensions of the hole, coating the surface with hot asphalt having a melting point between 140° and 180° fahr., and finally covering with chips varying in size from $\frac{1}{8}$ to $\frac{1}{2}$ in. in diameter, which should be rolled or thoroughly tamped.

The bituminous macadam built since June, 1908, has been constructed in accordance with the following agreement and specifications, which were compiled by Henry B. Drowne, Assoc. M. Am. Soc. C. E., in conjunction with the writer. The contracts were awarded on the basis of the construction of an ordinary macadam road, in accordance with the Standard Specifications of the State Board of Public Roads:

STATE OF RHODE ISLAND AND PROVIDENCE PLANTATIONS.

STATE BOARD OF PUBLIC ROADS.

AGREEMENT AND SPECIFICATIONS

For Extra Work, on Account of Construction of Bituminous Macadam Surface on a Section of State Highway in the Town of........ ...., County of................, State of Rhode Island.

THIS AGREEMENT, made and entered into this..................of ...., one thousand nine hundred and........, by and between the State of Rhode Island by the State Board of Public Roads, party of the first part, and.., party of the second part;

WITNESSETH, that the said party of the second part agrees with the said party of the first part to do all the work and furnish all the material (not herein agreed to be furnished by the party of the first part) to construct and complete ready for use the bituminous surface on a section of State Highway in the Town of...................., County of..................., State of Rhode Island, in accordance with and as described in the specifications herein contained, and in full accordance with the terms of this agreement:

That the said party of the second part further agrees with the

said party of the first part to furnish labor in such quantities as may be desired by the engineer at the following prices: Mr. Blanchard.

Common laborer..................	at	per hour,
Labor competent to work and handle bituminous products...........	at	per hour,
Single team and driver............	at	per hour,
Double team and driver...........	at	per hour.

That the said party of the second part further agrees with the said party of the first part to deduct from the total labor cost of each gang engaged in the laying of the stone mixed with the bituminous compound for each 10-hour day, or its equivalent:

1 common laborer, 10 hours.......	at	per hour,
1 watering cart hours.......	at	per hour.

That the said party of the first part agrees to pay the said party of the second part the costs of labor at aforesaid prices on the extra work directly connected with laying the bituminous macadam surface, plus 15 per cent.

That the said party of the second part further agrees with the said party of the first part to furnish tar kettles and accessories, including rakes, shovels, dippers, mixing boards, axes, brooms, pails, etc., and the said party of the first part agrees to pay the said party of the second part for the use of aforesaid utensils........ cents per day for each kettle actually used when the bituminous surface is being laid.

That the said party of the second part agrees with the said party of the first part to furnish cord wood at........per cord.

That the said party of the first part agrees with the said party of the second part to furnish all the bituminous material.

That it is mutually agreed that the party of the first part reserves the right to change at any time the method of construction; the bituminous compound; to require that the stone and bituminous compound shall either be mixed by hand or machine; to require that the stone shall be heated. The said party of the first part to supply the machines or driers if used.

In witness whereof, the parties to these presents have hereunto set their hands the year and date first above written.

* * * * * *

State of Rhode Island and Providence Plantations.

State Board of Public Roads.

Specifications for Bituminous Macadam.

These specifications are supplementary to and an integral part of the Standard Specifications, of the State Board of Public Roads, for macadam roads, and in no way replace them, except in so far as the construction of the No. 2 course and finished surface is concerned.

Plant.—The roller used in rolling the bituminous macadam surface shall not weigh more than 10 tons.

The kettles shall be of such shape and size that they can be easily transported by hand from point to point. No kettle, however, shall be

Mr. Blanchard. used that will not hold 2 bbl. of tar, except by special permission of the engineer.

When mixing on the road, a mixing board shall be provided for each gang so engaged. The mixing board shall be made of 2-in. plank, 16 ft. long, in two sections each 4 ft. wide.

Bituminous Compounds.—Coal-tar shall be placed in the kettles so that the depth of tar is not more than 18 in., unless otherwise directed by the engineer. Water in the tar shall be caught when the tar is being run into the kettle. If any rises to the surface of the tar in the kettle it shall be skimmed off. When tar is the only bituminous compound used, it shall be heated for 2 hours at a temperature between 150° and 180° fahr., then raised to at least 200° before using.

Kettles shall be placed so that hot tar will not have to be carried more than 50 ft.

In storing tar, barrels shall be laid on their sides with the bungs up.

Asphalt shall be placed in the kettles in such quantities as may be desired, and brought to a heat of 350° fahr. before using.

When the bituminous compound consists of tar and asphalt mixed half and half, the tar shall be heated in one kettle and the asphalt in another. The tar shall be brought to 200° fahr. before using; the asphalt to 350° fahr. before using. In mixing by hand the tar shall be placed on the stones before the asphalt.

Stone.—The No. 2 stone must be dry and free from dust. No No. 1 stone will be allowed in the No. 2 course.

No work shall be done when the No. 1 course is wet. The work shall not be resumed until the stone has dried out to the satisfaction of the inspector.

Mixing Bituminous Compound.—The bituminous compound shall be mixed with the stone either by hand or by machine until the stone is thoroughly coated to the satisfaction of the engineer or inspector. One and one-quarter gallons of bituminous compound shall be used per square yard of surface, unless otherwise directed by the engineer.

The No. 2 stone shall be laid to a thickness of 3 in. and rolled to 2 in.

The No. 2 course shall be rolled as directed by the engineer or inspector.

Painting.—After the surface has been thoroughly rolled and shaped up, to the satisfaction of the engineer or inspector, the surface shall be swept clean. The bituminous compound shall then be spread on the surface either by brooms or mops. Six-tenths of a gallon of the bituminous compound shall be used per square yard of surface, unless otherwise directed by the engineer.

Dust.—Dust screenings shall be perfectly dry.

Dust screenings shall be put on to a thickness not exceeding ¼ in., unless otherwise directed by the engineer or inspector.

No dust shall be put on until the surface has set to the satisfaction of the engineer or inspector.

When the surface is required to be painted, the dust screenings shall be put on immediately after the flush coat of bituminous compound has been applied.

The surface shall be rolled as directed by the engineer or inspector. **Mr. Blanchard.**

The section of State Road in the Town of East Providence referred to in the opening discussion was built by the mixing method with the bituminous mixture omitted from the surface of the No. 1 course. In the construction of this work, which was 11 870 ft. in length, various bituminous mixtures were used to meet conditions as they arose. The grading required for the formation of the gravel subgrade was light, throughout the entire length of the road, varying from a cut of 5 in. to a fill of 7 in. The broken stone used was a mixture of chlorite gneiss and indurated sandstone.

From Sta. 0 to Sta. 1950 on the first mile, Providence gas-house tar was used as the binding material. Between the stations mentioned the grade varied from a minimum of 0.36% to a maximum of 2.30 per cent. This section was constructed during June and July, the temperature varying from 55° to 95°, the average being 70° during working hours. The tar used was purchased from the Providence Gas-Works, and cost at the works $2.50 per bbl. The cost of the barrel per shipment of tar was $0.08, based on the cost of coopering and depreciation, as the State Board purchased a large number of barrels at the opening of the construction season. Loading and hauling full barrels and reloading and hauling empty barrels a distance of five miles cost $0.30 per bbl. The total cost of the tar, therefore, was $2.88 per bbl. The mixing and spreading gang included two tar men at $2.00 per day of 10 hours, and four common laborers at $1.50 per day. The cost of the labor, including 15% profit for the contractor, and based on an average rate of progress of 250 sq. yd. per day, was $0.039 per sq. yd. The wages of one common laborer who would have been employed on the No. 2 course if an ordinary macadam road had been constructed, was deducted, as per agreement, from the labor cost. The cost of accessories, which included rent of kettles, etc., and cost of cord wood, was $0.009 per sq. yd. The cost of watering, to lay the dust on the car tracks adjacent to the road, was $0.006 per sq. yd. The cost of the tar, at 1.25 gal. per sq. yd., was $0.072. The rebate accruing, as per agreement, by not using a watering cart on the construction of the macadam surface, was $0.004 per sq. yd. The total cost of the tar-macadam, in excess of the cost of ordinary macadam, therefore, was $0.122.

Although great care was exercised in watering, considerable dust, raised by car and other traffic on double tracks adjacent to the road, collected on the surface of the No. 2 course before it could be rolled and covered with screenings. The natural result was that the bond between the screenings and the No. 2 course was not as good as in the case of the tar-macadam in Charlestown and Narragansett. As the East Providence road is subjected to a heavy motor-car traffic, and

Mr. Blanchard. also to an unusually heavy horse-drawn vehicle traffic, it was decided, from the standpoint of insurance against disintegration, to seal the surface by painting the No. 2 course just before the application of the screenings. The result has been very satisfactory, and, it is believed, has justified the additional expense. The cost of tar, at 0.6 gal. per sq. yd., was $0.035. One tar man at $2.00 per day of 10 hours, and one common laborer at $1.50 per day, constituted the force. As an average of 390 sq. yd. was painted per day, the cost of the labor, including 15% profit for the contractor, was $0.01 per sq. yd. The cost of accessories, including kettles, dippers, pails, brooms, and fuel, was $0.004 per sq. yd. The total cost of painting with tar was $0.049 per sq. yd. Hence the total cost of the tar-macadam painted with tar, above the cost of ordinary macadam, was $0.171 per sq. yd. The above form of construction was used from Sta. 1950 to Sta. 2350.

A short distance beyond Sta. 2350, the road was narrow, leaving room for only one car track, whereas, up to this point, double tracks on one side of the road carried the heavy car traffic to summer colonies and to three amusement resorts. When the road widened out again, at Sta. 3500, the traffic was carried by two tracks, one on each side of the road. As it was impossible to close the highway to motor and other traffic, it was evident, due to the above arrangement of tracks, that the bituminous macadam would be subjected to more or less traffic before it set up. Due to these conditions and to the fact that the asphalt-tar-macadam constructed as an experiment in Narragansett was giving excellent service, it was deemed advisable to adopt a bituminous binder of 50% Texaco asphalt, Grade J, and 50% tar, which would accelerate the setting of the bituminous macadam and permit rolling within 12 to 24 hours. The asphalt-tar-macadam was used from Sta. 2350 on the first mile to Sta. 400 on the second mile, and from Sta. 1625 to Sta. 3238 on the second mile. On these sections the grade varied from a minimum of 0.12% to a maximum of 2.40 per cent. The work was done in July to October, the temperature varying from 31° to 89°, the average during working hours being about 60 degrees. The asphalt cost $2.66 per package of 40 gal. at the Providence plant of the Texas Company. The cost of loading and hauling a distance of 5 miles was $0.25 per package, making the total cost $2.91. As the total amount of bituminous material used in the mixing was 1.25 gal. per sq. yd., and as 50% of asphalt and 50% of tar were used, the cost of the asphalt was $0.046 per sq. yd., while the cost of the tar was $0.036. The mixing and spreading gang included two tar men at $2.00 per day of 10 hours, and five common laborers at $1.50 per day. Due to the fact that asphalt is more difficult to handle and sets more quickly than tar, the rate of progress per day was slower, averaging 200 sq. yd. The cost of the labor, therefore, was $0.058 per sq. yd. The cost of accessories was $0.009, the cost of watering to allay the dust was

$0.006, and the rebate on watering was $0.005 per sq. yd. The total excess cost of the asphalt-tar-macadam, without the surface coat, was $0.15 per sq. yd. The complete excess cost, including the surface coat of 0.6 gal. of tar, was $0.199 per sq. yd. As an experiment, the surface coat of tar was omitted for a section of 100 ft. Also, as an experiment, a section of about 200 ft. was painted with Texaco asphalt, Grade J, in place of tar. It was found that 0.6 gal. of asphalt would cover 1 sq. yd., hence, the cost of the bituminous material was $0.044 per sq. yd. As the same number of yards were painted per day as when tar was used, the cost of labor, etc., was the same, the total cost, therefore, was $0.058 per sq. yd., making the total cost of the asphalt-tar-macadam, painted with asphalt, $0.208 per sq. yd. Mr. Blanchard.

The approach of cold weather necessitated another change in the bituminous material used. It was found to be practically impossible to coat the stone thoroughly, using asphalt and tar as described above. Due to this fact, and also because motor, horse-drawn vehicle, and car traffic had fallen off 50%, it was decided to use tar as the bituminous material in mixing the No. 2 course, and Texaco asphalt, Grade H, for painting the surface. The cost of the tar-macadam, omitting the surface coat, was the same as described above, namely, $0.122 per sq. yd. As the Grade H asphalt cost $2.94 per package at the plant, and loading and hauling cost $0.25 per package, the total cost was $3.19 per package. The cost of 0.6 gal. used per square yard, therefore, was $0.048, making the total cost of the painting $0.062 per sq. yd. The total cost of the bituminous macadam, therefore, was $0.184 per sq. yd.

From Sta. 400 to Sta. 1625 on the second mile, an experimental section was built, using Providence Gas-Works tar and Genasco Road-Compound Base, furnished, without expense to the Board, by the Barber Asphalt Paving Company. These materials were used in the proportion of 25% Genasco Base and 75% tar. The method of construction was similar to the methods already described, the surface being painted with tar.

Judging from experience acquired during the construction of bituminous macadam in Rhode Island in 1906, 1907 and 1908, it is apparent to the writer that the field of experimentation for the season of 1909 should include the development of economical mixing machinery, the determination of the economics and efficiency of heated stone, and more complete laboratory and field analyses of bituminous materials.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

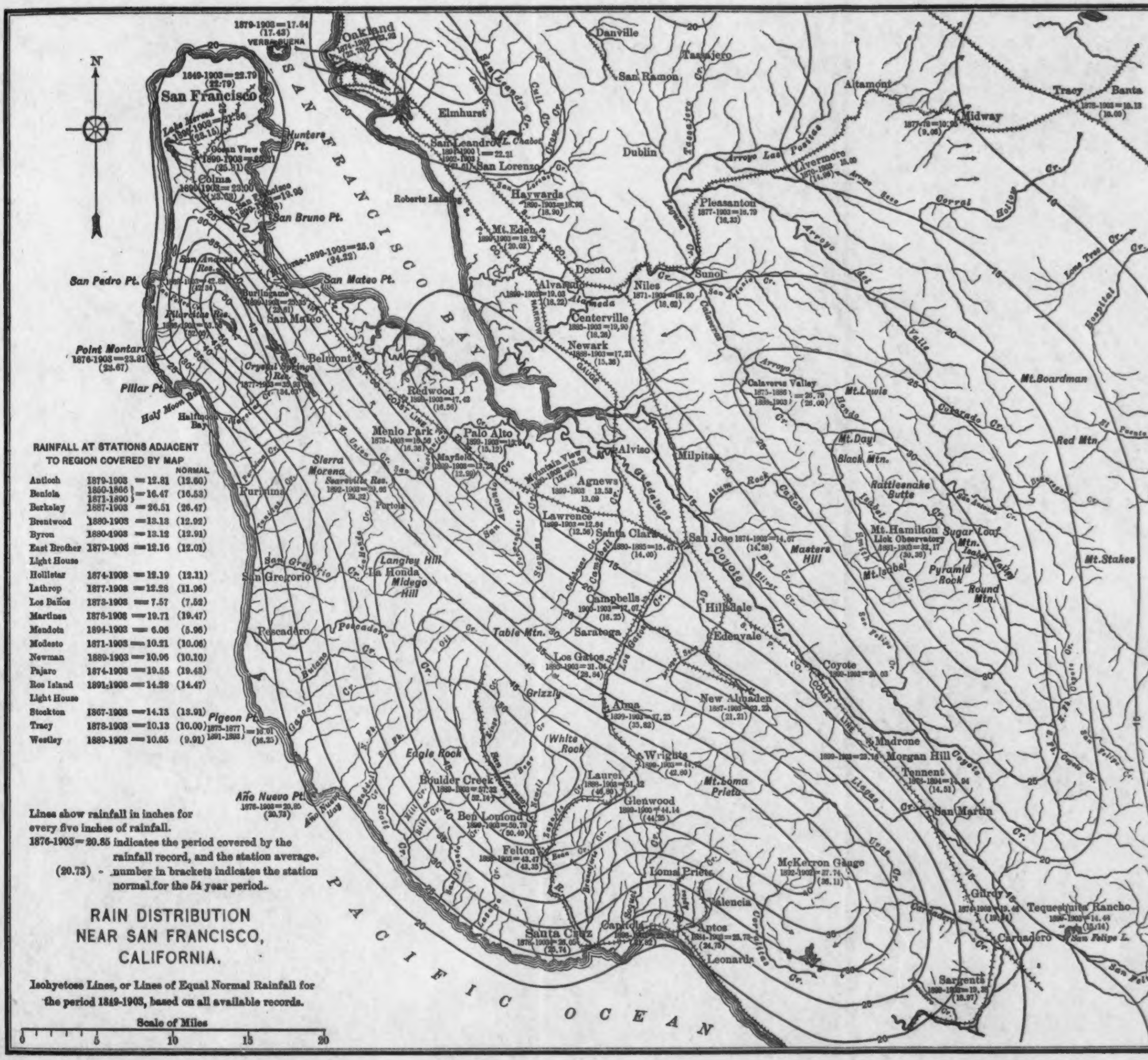
Paper No. 1090

RAIN AND RUN-OFF NEAR SAN FRANCISCO, CALIFORNIA.

By C. E. Grunsky, M. Am. Soc. C. E.

With Discussion by Messrs. E. C. Murphy, Edwin Duryea, Jr., H. L. Haehl and A. C. Toll, A. G. McAdie, and C. E. Grunsky.

In 1903 a study of the availability, as water producers, of a number of water-sheds near San Francisco, California, made an analysis of rainfall and run-off conditions necessary. Studies were made of the distribution of rain throughout that part of the Coast Range lying between the latitudes of Santa Cruz and San Francisco, and extending eastward from the Pacific Ocean to the westerly edge of the San Joaquin Valley. The result of these studies is shown on the map, Plate LIX. Thereupon, a computation was made of the water production in three basins tributary to the three peninsula reservoirs of the Spring Valley Water Company, which furnish water for San Francisco. The records of the water yield of these three storage reservoirs, Pilarcitos (elevation 680 ft.), San Andreas (435 ft.), and the Crystal Springs (280 ft.), are more than ordinarily reliable, and, fortunately, cover a long time-period. Ever since these reservoirs or any of them have been in service (1865) Mr. Hermann Schussler has been the Chief Engineer for the Spring Valley Water Company, and his records of water consumption from each reservoir, and of the stage


RAINFALL AT STATIONS ADJACENT
TO REGION COVERED BY MAP
NORMAL
Antioch 1879-1903 = 12.81 (12.60)
Benicia 1850-1866 / 1871-1890 = 16.47 (16.53)
Berkeley 1887-1903 = 26.51 (26.47)
Brentwood 1880-1903 = 13.13 (12.92)
Byron 1880-1903 = 13.12 (12.91)
East Brother Light House 1879-1903 = 12.16 (12.01)
Hollister 1874-1903 = 12.19 (12.11)
Lathrop 1877-1903 = 12.28 (11.96)
Los Baños 1873-1903 = 7.57 (7.52)
Martinez 1878-1903 = 19.71 (19.47)
Mendota 1894-1903 = 6.06 (5.96)
Modesto 1871-1903 = 10.21 (10.06)
Newman 1889-1903 = 10.96 (10.10)
Pajaro 1874-1903 = 19.55 (19.43)
Roe Island Light House 1891-1903 = 14.28 (14.47)
Stockton 1867-1903 = 14.13 (13.91)
Tracy 1878-1903 = 10.13 (10.00)
Westley 1889-1903 = 10.65 (9.91)
Lines show rainfall in inches for every five inches of rainfall.
1876-1903 = 20.85 indicates the period covered by the rainfall record, and the station average.
(20.73) - number in brackets indicates the station normal for the 54 year period.
RAIN DISTRIBUTION
NEAR SAN FRANCISCO,
CALIFORNIA.
Isohyetose Lines, or Lines of Equal Normal Rainfall for the period 1849-1903, based on all available records.
Scale of Miles
0 5 10 15 20
N
San Francisco
1849-1903 = 22.79 (22.79)
SAN FRANCISCO BAY
PACIFIC OCEAN
Oakland
Verba Buena
Elmhurst
San Leandro
San Lorenzo
Hayward
Danville
San Ramon
Tassajero
Dublin
Altamont
Tracy
Banta
Midway
Livermore
Pleasanton
Sunol
Niles
Decoto
Alvarado
Centerville
Newark
Hunters Pt.
San Bruno Pt.
San Mateo Pt.
Ocean View
Colma
San Pedro Pt.
Burlingame
San Mateo
Point Montara
1876-1903 = 23.81 (23.67)
Pillar Pt.
Half Moon Bay
Belmont
Redwood
Menlo Park
Palo Alto
Mayfield
Mountain View
Alviso
Milpitas
Agnews
Lawrence
Santa Clara
San Jose
Masters Hill
Calaveras Valley
Mt. Lewis
Mt. Boardman
Red Mtn.
Mt. Day
Black Mtn.
Rattlesnake Butte
Mt. Hamilton
Lick Observatory
Sugar Loaf Mtn.
Mt. Isabel
Pyramid Rock
Round Mtn.
Mt. Stakes
Purisima
Sierra Morena
Searsville Res.
Portola
San Gregorio
La Honda
Langley Hill
Midego Hill
Pescadero
Table Mtn.
Campbells
Saratoga
Hillsdale
Edenvale
Los Gatos
Coyote
New Almaden
Alma
Grizzly
White Rock
Pigeon Pt.
Eagle Rock
Boulder Creek
Wrights
Laurel
Mt. Loma Prieta
Madrone
Morgan Hill
Tennent
San Martin
Año Nuevo Pt.
1876-1903 = 20.85 (20.73)
Año Nuevo Bay
Ben Lomond
Glenwood
Felton
McKerron Gauge
Loma Prieta
Valencia
Santa Cruz
Capitola
Aptos
Leonards
Gilroy
Tequesquita Rancho
Carnadero
San Felipe L.
Sargents
Orestimba Cr.

of water in each, have been used in estimating the annual water production of the three drainage basins.

A comparison of the annual water yield of the several basins with the annual rainfall furnished reliable data on which to base general conclusions relating to run-off in Coast Range areas. By drawing upon other information, as will be explained hereinafter, applying to larger and higher mountain areas, it was also possible to forecast run-off quantities for such regions as the Sierra Nevada Mountains. The results of these studies and the method of utilizing the available records, should prove of interest to engineers who have to deal with similar problems.

Rain does not fall in California in every month of the year, as in the Eastern States. The rainy season begins in November and ends in April. So little rain falls from May 1st to the end of October that this period may be called rainless. There is no rain during this period which has any effect worthy of note upon the flow of streams.

Throughout the State, however, there is great variation in the normal annual rainfall, and, in the coast and central valleys, this is generally from 15 to 30 in. It rises to more than 70 in. in the Sierra Nevada Mountains, 150 miles northeasterly from San Francisco, and to more than 90 in. in the extreme northwesterly portions of the State; it is only 10 in. at some points of Sacramento Valley, and less than 6 in. in parts of San Joaquin Valley; it drops to only 2 to 3 in. in the Cahuilla (Salton) Basin. One feature, however, is especially noteworthy. The rain storm is ordinarily an atmospheric disturbance of large extent. It is not of the same nature as eastern thunder storms, but is of the general type of winter storms which, in the East as in the West, sweep over vast areas. Owing to the wide distribution of rain in ordinary rain storms, and to the freedom from local storms, the rainfall records at single stations are better indices of the amount of precipitation on large tracts than is ordinarily the case for records of rain in the East and in the Middle West.

With a view of illustrating the breadth of the storm area, it may be stated that the same atmospheric disturbance which brings rain to the Pacific Coast northerly from California, also brings rain to (or threatens with rain) all northern and central parts of California as far south as Tehachapi, where a mountain spur connects the Coast Range with the Sierra Nevada Mountains. As a rule, the greater the

fall of rain at central points of this storm area the greater the surface extent of any cyclonic disturbance. The recurrence of rain storms (from six to twenty in a rainy season of 6 months) has the usual equalizing effect of repetition, and thereby increases the probability that the fall of rain in the course of a year, at any point of the central and northern portions of California, will bear a fairly uniform relation to the rainfall at some central point of observation, such as San Francisco or Sacramento. Exceptions to such a law are sure to occur, and have occurred. A notable exception was the rain distribution in 1867-68, in which an abnormally heavy fall of rain in the mountain region tributary to the San Joaquin Valley was not indicated by the rainfall conditions of that year at points in latitudes northerly from San Francisco.

Among the interesting facts that have come to the attention of those who are familiar with the rainfall records in California is the following, which seems to be true wherever the rainfall exceeds 10 in. per annum. The maximum annual rain at any point is twice the normal, and the minimum is two-fifths of the normal. (Marsden Manson, M. Am. Soc. C. E., now City Engineer of San Francisco, puts the latter at one-third.) This relation of maxima and minima to the normal is of great value in the discussion of the water productiveness of a region in which there is scant information obtainable relating to rainfall.

One of the early observations made by the engineers who from time to time have been consulted on the subject of an adequate water supply for San Francisco was the recurrence in succession of so-called dry winters, that is, of rain years with a fall of rain so far below normal as to be classed as minimum years. Two such years now and then follow each other, and there may be a series of years, up to about ten, of which none materially exceeds the normal. As the years of minimum rainfall produce practically no run-off from areas near San Francisco, and years of normal rainfall only a moderate amount, the conclusion was reached that the storage capacity when compared with the run-off from the area tributary to a reservoir should be relatively large, and that the aggregate storage capacity should be equivalent to about 900 days' supply. This idea was first set forth clearly by the late Colonel G. H. Mendell,* M. Am. Soc. C. E. It applies, of course,

* San Francisco Municipal Reports, 1876.

only as long as dependence is placed upon small near-by water-producing areas having the character above described, which are entirely unproductive in seasons of light rainfall.

Probably the first attempt to construct isohyetose curves for the entire State of California, was made by the writer in 1886, under the direction of State Engineer William Ham. Hall, M. Am. Soc. C. E. More than 200 station records were available, many being at railroad stations. Nearly all the railroad station records dated from 1870. It was on account of the commencement of a majority of the rain records in that year that the writer determined to let the isohyetose curves represent the mean annual precipitation for the period subsequent to 1870, in fact, for the 14 years, or rain seasons, 1870-71 to 1883-84. All records that covered only a fraction of this 14-year period were corrected by comparison with one, two, or three near-by complete station records. Thus, for example, if a record at Station *A* covered only the last 6 winters of the period, the mean rainfall for these 6 years was compared with the rainfall in the same 6 years at Stations *B, C,* and *D.* The ratios established by this comparison were then applied to the means for the whole period at *B, C,* and *D,* and gave three values for rain at *A.* These were then averaged or combined, with unequal weights, as each case seemed to require, to get the most probable precipitation at *A* for the full period. The curves of equal precipitation were then drawn in the same way as contour lines. The record for the selected 14 years was not greatly at variance with the normal rainfall. The resulting map* is subject to correction, however, particularly throughout a large extent of the Sierra Nevada Mountains, where no records of rain were obtainable at the time it was prepared.

This study of 1886 made it quite apparent that the orographic features of the State were potent factors in modifying the rainfall, and were therefore material aids in extending the rain curves over the areas where rainfall records were lacking.

Testimony can be borne, with no small degree of satisfaction, to the general reliability of the many records of rainfall which have entered into the special study of 1903. A careful examination of the isohyetose curves on Plate LIX, and a comparison of the normal rain, which they indicate, with the individual station records will

* Published in Irrigation Development, Part II, Irrigation in California, William H. Hall, State Engineer, 1886.

show, not only that the curves did not have to be forced, that they fall naturally into the positions assigned to each, but also that there are only two station records—at Coyote and at Morganhill—which, when expanded from 4 years to the 54-year period, do not fit in with records at other near-by stations nor with the isohyetose curves.

It may be stated as a general proposition that station records should be accepted as representing correctly what has happened at the point of observation, and that records which do not harmonize with the supposed facts should be discarded only when there is corroborative evidence of their unreliability. This is particularly true of all observations conducted under the direction of the United States Weather Bureau, and applies even in those cases where proximity of the rain gauge to high buildings, unfavorable exposure on roofs of buildings, and other disturbing causes lead to the conclusion that the station record is at times in error. No station is ever located so that such errors would be constant for all possible directions of the wind and wind velocities. Moreover, notwithstanding occasional error, the station record may still be an excellent index of what is happening, in the way of precipitation, throughout broad areas.

This subject should not be passed by without pointing out the unquestioned advantage that would result if the Weather Bureau would supplement all its records taken in large cities, or under otherwise unfavorable conditions, by records on exposed ground in the surrounding country. Such supplemental records should be primarily for the purpose of ascertaining precipitation aggregates, and frequency of observation at such controlling stations, therefore, would not be requisite.

It is particularly noteworthy that in all studies of run-off within the Pacific Coast States, the seasonal division of time should not run with the calendar year. For the sake of uniformity, the records of total rain published in most Federal official documents have been for each calendar year for the Western States, just as they have been for the Eastern States. This system of publication, if annuals alone are considered, does not enable one to make a correct distinction between the wet year and the dry year. The rainfall record for a year should include only one rainy season, and not parts of two. Moreover, as the precipitation in each rainy season is followed by a rise in the streams, and as the winter snows which feed the streams do not disappear from

the high mountains until late in the summer, the natural beginning of a rain and run-off year would be about September 1st. It will make practically no difference for areas in California if this date is shifted to October 1st or even to November 1st. When precipitation is noted for the rain year, 1870-71, it means the precipitation in a year including completely the winter of 1870-71. A late date for the commencement of the rain year is desirable in order that it may run with the year of stream flow. The latter should cover a complete cycle from one annual low stage of the rivers to the next annual low stage, and, for California, may begin at any time from September 1st to November 1st. Owing to the fact, however, that the fall of rain from May 1st to November 1st is trifling, it would not be seriously objectionable to let the rain year and the run-off year overlap to some extent.

Within the area covered by the rainfall map, Plate LIX, nearly 5 000 sq. miles—practically square in outline—there are 56 points at which rainfall records had been kept for some years prior to 1903. In this statement the records at San Francisco are counted as one. The longest record is that of San Francisco which was commenced in 1849 by a careful observer, Mr. Thomas Tennent, maker of nautical instruments. Later, the number of records in that city increased, at one time to five. All are accepted as being of equal weight with the record of the Signal Service, beginning in 1871, and, later, the record of the U. S. Weather Bureau. Special weight cannot be given to the rain record of the Weather Bureau, because it applies to several stations successively occupied, and suffers from disturbing causes, as do all records on the roofs of buildings. For San Francisco, therefore, the several records were combined, and the average was accepted as the best indication of the probable amount of rain that fell in the city. The mean annual rainfall, or the normal on the 54-year basis, 1849-1903, was found from the composite record to be 22.79 in. for San Francisco.

With the San Francisco record there were compared, for corresponding periods of time, the records at San José and at Oakland, 1874-1903; Mt. Hamilton (Lick Astronomical Observatory), 1881-1903; Santa Cruz, 1878-1903; Pilarcitos, 1866-1903; San Andreas, 1869-1903; Crystal Springs, 1877-1903, and others. By using the ratios thus ascertained to exist between the rainfall at San Francisco and that at

each of these points, and applying this ratio to the normal rain at San Francisco, a value of the probable normal for the 54-year period at each of these points was ascertained. For the most important stations, such as Mt. Hamilton, San José, and Santa Cruz, the rainfall for missing years of the 54-year period was approximated. These secondary stations, thereupon, were used in the computation of the normal precipitation for the 54-year period at each point where any part of the time was covered by a local record. The method used is that just explained.

It remains to be added that, in addition to the stations noted on the map, there were records at 18 points, to the north and to the east of the area covered, taken into account in the construction of the rain curves.

The topographic features shown on the map require some explanation. San Francisco lies at the northerly extremity of the San Mateo Peninsula. A spur of the Coast Range forms the backbone of this peninsula. To some extent it is subdivided into secondary ridges, but all is mountainous except a narrow fringe along the westerly side of San Francisco Bay. Within San Francisco the highest point, Twin Peaks, is more than 900 ft. above sea level. To the southward of San Francisco, cutting from northwest to southeast obliquely across the narrowest part of the peninsula, are the San Bruno Mountains, having a maximum height of 1 300 ft. This range is separated from the southerly extension of the peninsula ridge by a gap barely more than 200 ft. above the sea. The peninsula mountains are soil-covered, their summits are rounded, and their slopes moderate. Much of the ground is brush-covered, some is timbered, and comparatively little bare rock is exposed. Elevations on the ridge rise to several thousand feet midway between San Francisco and Santa Cruz, and to 3 850 ft. at Loma Prieta, 12 miles northeasterly from Santa Cruz. To the eastward of the peninsula mountains is San Francisco Bay and the Santa Clara Valley, with its southerly continuation, the Gilroy Valley. The two valleys merge into each other, the highest land between them being about 300 ft. above the sea. These two valleys and the San Francisco Bay separate the peninsula spur of the Coast Range from a series of parallel spurs of which one culminates at Mt. Hamilton, slightly more than 4 000 ft. in altitude, and another at Mt. Diablo, of similar height, and just to the north of the easterly portion of the

area covered by the map. The crests of several of this group of Coast Range spurs are at elevations of from 2 000 to 3 000 ft. The lowest gap across the range eastward from San Francisco is at Altamont, about 700 ft. in elevation. Another low point is Pacheco Pass, just off the map near its southeast corner.

The computed station normals for the 54-year period, 1849-1903, were used as a basis for the construction of the isohyetose lines. These conform in a broad way to the contour lines of the country, that is to say, they hold the same general direction. It will be noticed, however, that the maximum rainfall is not at the point of maximum elevation, but, as a rule, lies upon the southwest slopes of the mountain chains. On the peninsula mountains a maximum is noted just west of the crest line. Near Pilarcitos Reservoir it very nearly coincides with the crest, because the range there is narrow, and if there is a maximum westerly from Pilarcitos it would be disclosed only by additional rain record stations. The position of the rainfall maximum on the southwest slope of the range is pronounced near Laurel and Boulder Creek, some miles north of Santa Cruz. Eastward from Santa Clara Valley the region in which rainfall is a maximum is well up on the range, about midway in distance between the valley and the crest line which divides its waters from those of San Joaquin Valley.

Taking a course from a point on the ocean about 12 miles northwest from Santa Cruz in a northeasterly direction toward Mt. Hamilton, the normal annual rainfall increases from 20 in. on the water to 25 in. at the shore line, and to 50 in. at about two-thirds of the distance from the ocean to the mountain crest. Continuing on the northeasterly course, there is a decrease to less than 15 in. in the Santa Clara Valley, and thereupon an increase to 31 in. at a point just east of Mt. Hamilton, which, as already stated, is about half way from Santa Clara Valley to the range crest. Thence to San Joaquin Valley the rain decreases to about 10 in.

Evidently, the moisture-laden air, as it rises up the mountain slopes, becoming cooler and losing density with increasing altitude, is forced to part with so much moisture before it reaches the mountain crest that the northeast slopes and the valleys beyond receive much less precipitation than the opposite mountain slopes.

This same phenomenon is equally marked in the Sierra Nevada Mountains. The rain at the southwesterly base of these mountains

in Sacramento Valley is about 20 in. per annum. It is more than 70 in. at high altitudes on the southwest slope of the mountains, at points northeastward from Sacramento Valley, and it drops to only 4 or 5 in. on the plains of Nevada, from 4 000 to 5 000 ft. in altitude, east of the range.

The curves of normal rainfall, particularly where there is such pronounced change in amount within a few miles, as may be seen in parts of the mapped area, have served admirably as the basis for computing the normal fall of rain upon any water-shed. The normal rainfall amounts thus established for the areas tributary to Pilarcitos Reservoir, to San Andreas Reservoir, and to Crystal Springs Reservoir were thereupon used as aids in approximating the annual precipitation upon each of these water-sheds.

The process may be briefly stated. The three rainfall records, at Pilarcitos, at San Andreas, and at Crystal Springs, after being expanded, as already explained, to the full 54-year period, were combined by averaging the rain-year annuals. In this way a composite rain record for the reservoir areas, from 1866 to 1903, was obtained, and this was used as an index of the relative amount of rain that fell in each rain year. The composite normal for the 54 years varied from the normals in each of the three basins, and the extent of this variation was ascertained for each. Thereupon, a correction of the composite annuals was made for each basin, and the corrected values were accepted as the annual precipitation on the basin areas. The three records were combined, as here explained, instead of being used separately, because they are for areas which are close to one another, and it is believed that such a composite record is a somewhat more reliable index of the rain on a considerable extent of country than either of the three records used alone would be.

As it is desired to know for each of the three reservoir drainage basins what quantity of water annually flowed from land areas, exclusive of water surface, the following procedure may be adopted.

The water consumption from each reservoir is known from the records of the Spring Valley Water Company. The stage of water, and therefore the annual increase or decrease in the quantity of water stored, is also known for each. Also, approximately at least, the quantity of water which had to be allowed to flow to waste.

Let R = rainfall per annum, in inches,

A = drainage area, in square feet (including the reservoir surface),

a = area of reservoir water surface, in square feet,

E = evaporation per annum, in inches,

U = useful water consumption per annum, in gallons,

W = waste per annum, in gallons,

r = run-off from land only, in inches,

S = storage increase per annum, in gallons.

Then $\frac{r}{12}(A - a) = \frac{S}{7.5} + \frac{E}{12}(a) + \frac{U}{7.5} - \frac{R}{12}a + \frac{W}{7.5}$

$$r = \frac{1.6\,(S + U + W)}{A - a} + \frac{a}{A - a}(E - R)$$

For $E = 48$ in. this becomes

$$r = \frac{1.6\,(S + U + W)}{A - a} + \frac{a}{A - a}(48 - R).$$

The evaporation from the water surface of the reservoirs was assumed at 48 in. per year. It is believed that this assumption is liberal. Some error in this assumption would have but small effect upon the conclusions herein reached relating to run-off. The above value was assumed because it is the amount indicated by a 4 years' series of observations on Kings River at Kingsburg in the San Joaquin Valley.* The mean annual temperature at the reservoirs is probably somewhat lower than that at Kingsburg, and the greater proximity of the reservoirs to the Pacific Ocean may also act to keep the evaporation lower than in the interior valley. If there is error in the assumption, it is therefore probable that it is in the direction of too much evaporation.

It remains to be stated that all three of these reservoirs are of relatively large capacity when compared with the small tributary areas. There are, therefore, many seasons in which there is no waste, in which all water is caught. This is particularly true of Crystal Springs Reservoir, which has been full only twice. There is, moreover, a certain interdependence between the reservoirs. The relation in which the Pilarcitos Reservoir stands to the San Andreas Reservoir is so close, in fact, that it has seemed advisable to combine the two

* "Physical Data and Statistics," William Ham. Hall, State Engineer of California, 1886, page 378.

and treat them as a single reservoir in this discussion. The arrangement is such that the waste from the Pilarcitos flows to the San Andreas until the conduit capacity is exceeded, whereupon some water goes to the stream. Any excess of water received by the San Andreas Reservoir is in turn delivered to the Crystal Springs Reservoir.

TABLE 1.—ANNUAL RAINFALL IN THE DRAINAGE BASINS OF THE PILARCITOS, SAN ANDREAS AND CRYSTAL SPRINGS RESERVOIRS.

Year.	Reservoir composite.	Pilarcitos.	San Andreas.	Pilarcitos and San Andreas.	Upper Crystal Springs.	Crystal Springs.
1866-67	65.17	74.20	.....	....	....	....
67-68	81.77	92.90	.....	....	....	....
68-69	48.26	54.85	.....	....	....	....
69-70	43.13	49.00	42.45	45.3	....	....
1870-71	35.09	39.90	34.60	36.8	....	....
71-72	80.53	91.60	79.50	84.6	....	....
72-73	39.17	44.30	38.60	41.1	....	....
73-74	49.86	56.70	48.80	52.4	....	....
74-75	44.36	50.40	43.80	46.6	....	....
75-76	69.47	79.00	68.50	72.9	....	....
76-77	22.37	25.50	22.05	23.5	....	...
77-78	68.14	77.60	67.20	71.6	51.5	....
78-79	49.47	56.20	48.80	51.9	37.4	....
79-80	53.79	61.20	53.00	56.5	40.7	....
1880-81	48.77	55.50	48.10	51.2	36.9	....
81-82	31.28	35.60	30.90	32.8	23.6	....
82-83	30.21	34.40	29.80	31.7	22.9	....
83-84	50.22	57.20	49.50	52.7	38.0	....
84-85	35.02	39.90	34.50	36.8	26.5	....
85-86	49.29	56.10	48.60	51.8	37.3	....
86-87	34.31	39.10	33.80	36.0	26.0	....
87-88	33.67	38.30	33.20	35.4	25.4	....
88-89	37.45	42.70	33.90	39.3	....	31.8
89-90	70.83	80.70	69.70	74.4	....	60.1
1890-91	35.79	40.70	35.30	37.6	....	30.3
91-92	36.11	41.10	35.60	37.9	....	30.6
92-93	51.58	58.80	50.80	54.2	....	43.7
93-94	46.99	53.50	46.40	49.3	....	39.8
94-95	61.48	70.00	60.60	64.5	....	52.1
95-96	45.22	51.50	44.60	47.5	....	38.4
96-97	45.96	52.30	45.30	48.2	....	39.0
97-98	23.62	26.90	23.20	24.8	....	20.0
98-99	38.56	43.90	38.00	40.5	....	32.7
99-00	40.48	46.10	40.00	42.5	....	38.3
1900-01	40.14	45.70	39.50	42.1	....	34.0
01-02	37.07	42.20	36.60	38.9	....	31.5
02-03	35.57	40.60	35.20	37.5	....	30.3

Both Pilarcitos and San Andreas Reservoirs have had their natural water-sheds increased by the addition of small outside areas in which the run-off is caught by suitably-arranged conduits and led into the reservoirs. Such conduits do not at all times trap and deliver all the run-off water which during heavy rainfall often exceeds the conduit capacity. Consequently, such outside areas have been considered as being half as efficient as areas entirely within the several drainage

basins. Moreover, as outside areas have been added or cut out from time to time, the drainage basin areas have been subject to some change. The following have been introduced into the calculation:

Pilarcitos and San Andreas combined, 10.4 sq. miles until 1896, then 11.3 sq. miles until 1902, and finally 10.5 sq. miles.

Upper Crystal Springs, which later became a part of the Crystal Springs Reservoir, 13.79 sq. miles. Crystal Springs Reservoir (including Upper Crystal Springs) 23.16 sq. miles until 1896, thereafter 22.26 sq. miles.

TABLE 2.—TOTAL WATER PRODUCTION IN THE DRAINAGE BASIN OF THE COMBINED PILARCITOS AND SAN ANDREAS RESERVOIRS (INCLUDING WATER SURFACE).

Year.	Water consumption, in millions of gallons. *U.*	Storage increase, in millions of gallons. *S.*	Waste, in millions of gallons. *W.*	$U+S+W$, in millions of gallons.
1869-70	1 466	+ 511	....	1 977
70-71	1 693	− 775	....	918
71-72	2 483	+ 3 178	Large.	Uncertain.
72-73	2 689	− 385	....	2 394
73-74	3 118	+ 188	Probable.	3 306 +
74-75	3 532	− 1 519	....	2 013
75-76	3 957	+ 1 978	Some.	5 935 +
76-77	3 343	− 3 431	....	− 88
77-78	3 323	+ 3 215	Some.	6 537 +
78-79	3 800	− 284	....	3 516
79-80	3 944	+ 548	Probable.	4 492 +
1880-81	4 311	+ 705	....	5 016
81-82	4 635	− 2 410	....	2 225
82-83	3 265	− 1 423	....	1 842
83-84	3 453	+ 899	....	4 352
84-85	3 674	− 1 415	....	2 259
85-86	2 140	+ 2 991	....	5 131
86-87	3 455	− 690	....	2 765
87-88	3 304	− 1 061	....	2 243
88-89	1 987	+ 238	....	2 225
89-90	2 252	+ 4 366	Some.	6 618 +
1890-91	2 444	− 868	....	1 576
91-92	1 407	− 389	....	1 018
92-93	2 925	+ 176	Probable.	3 101 +
93-94	2 480	− 30	....	2 450
94-95	3 325	− 688	Large.	Uncertain.
95-96	3 777	− 1 260	....	2 517
96-97	4 286	− 452	....	3 834
97-98	3 113	− 1 729	....	1 384
98-99	583	+ 1 077	....	1 660
99-00	2 779	+ 295	....	3 074
1900-01	4 383	− 1 280	....	3 203
01-02	3 087	+ 215	....	3 302
02-03	3 871	+ 484	....	4 355

The water surface in the reservoirs varied in area from month to month and from year to year. For convenience, it has been assumed that the relation of the water-surface area of each reservoir to its tributary water-shed has been constant. The water surfaces of

Pilarcitos and San Andreas were taken at 7.5% of their combined drainage basin areas; Upper Crystal Springs Reservoir covered, at average conditions, about 5.5% of its drainage basin area, and Crystal Springs about 8 per cent.

The storage increase or decrease, as noted in Tables 1 to 4, is based on the water stage in the reservoirs on November 1st of each year. The tables have not been burdened with the details of the calculation. It is to be noted that, in the case of Pilarcitos and San Andreas Reservoirs, there is uncertainty relating to the quantity of water wasted. The values of run-off as noted are therefore small rather than large—a fact which may be taken into account in the construction of a run-off curve.

TABLE 3.—Total Water Production in Drainage Basins of Upper Crystal Springs and Crystal Springs Reservoirs (Including Water Surfaces).

Year.	Water consumption, in millions of gallons. U.	Storage increase, in millions of gallons. S.	Waste, in millions of gallons. W.	$U + S + W$, in millions of gallons.
1877-78........	0	+ 3 179	.....	3 179
78-79........	0	− 208	.....	− 208
79-80........	0	+ 154	.....	154
1880-81	0	− 297	.....	− 297
81-82........	28	− 57	.....	− 29
82-83........	629	− 648	.....	− 19
83-84........	76	+ 753	.....	829
84-85........	891	− 787	.....	104
85-86........	2 538	− 528	.....	2 010
86-87........	1 280	− 882	.....	398
87-88........	1 176	− 528	.....	648
88-89........	2 768	+ 868	.....	3 636
89-90........	2 966	+ 7 210	6 737	16 912
1890-91........	1 179	+ 1 802	.....	2 981
91-92........	1 691	− 670	.....	1 021
92-93........	2 019	+ 4 480	.....	6 499
93-94........	2 416	+ 1 076	.....	3 492
94-95........	2 611	− 40	4 674	7 245
95-96........	2 441	− 368	.....	2 073
96-97........	3 761	− 726	.....	3 035
97-98........	3 796	− 4 913	.....	− 1 117
98-99........	4 348	− 2 973	.....	1 375
99-00........	2 435	− 1 036	.....	1 399
1900-01........	1 531	− 697	.....	834
01-02........	1 626	− 1 361	.....	265
02-03........	825	+ 1 757	.....	2 582

For convenience, the data in Tables 1 to 4 may be combined by averaging all rainfalls within 5-in. limits and averaging the corresponding run-offs. This procedure gives the following results:

Pilarcitos and San Andreas drainage basins.

For	rainfall	24.1	in.,	the	run-off	is	5.7	in.;	with weight	2.
"	"	32.2	"	"	"	"	13.5	"	" "	2.
"	"	37.7	"	"	"	"	14.4	"	" "	9.
"	"	41.6	"	"	"	"	14.8	"	" "	4.
"	"	47.7	"	"	"	"	15.0	"	" "	5.
"	"	52.3	"	"	"	"	24.1	"	" "	6.
"	"	56.5	"	"	"	"	26.1	"	" "	1.
"	"	73.0	"	"	"	"	36.2	"	" "	4.

Upper Crystal Springs drainage basin.

For	rainfall	22.7	in.,	the	run-off	is	1.4	in.;	with weight	2.
"	"	26.0	"	"	"	"	3.0	"	" "	3.
"	"	38.0	"	"	"	"	3.0	"	" "	5.
"	"	51.5	"	"	"	"	13.8	"	" "	1.

Crystal Springs drainage basin (including Upper Crystal Springs).

For	rainfall	20.0	in.,	the	run-off	is	0	in.;	with weight	1.
"	"	31.6	"	"	"	"	6.0	"	" "	7.
"	"	38.9	"	"	"	"	7.6	"	" "	4.
"	"	47.9	"	"	"	"	18.4	"	" "	2.
"	"	60.1	"	"	"	"	44.4	"	" "	1.

Some of the water which falls to the earth in the form of rain or snow is absorbed by the soil or passes by infiltration into the porous fissured or cavernous sub-surface strata of the earth's crust. A small part of it enters into the structure of grasses, shrubs, and trees, another small part is lost by evaporation from the surface of leaves and other parts of plants, and from the grass cover of the ground when there is any. The remainder is visible run-off. That part of the water which sinks into porous or otherwise open or pervious substrata, flows therein to some point of outfall and is to be considered a part of the run-off in the sense in which the term is herein used. It is assumed in this discussion that all water thus moving underground and reappearing in springs shall have reached the surface, and shall have become visible run-off at the point for which run-off is estimated. All other water which sinks into the soil returns to the surface in the course of time and is consumed in sustaining plant life or is evaporated. On the assumption, therefore, that no material

part of the water from a drainage basin flows in subterranean channels past the point for which run-off is estimated, it may be broadly stated that all water falling as rain either appears in the stream as run-off or evaporates. The assumption here made, that there is little or no sub-surface flow, is justified in ordinary cases. Where conditions indicate the probability of flow in sub-surface channels or in porous strata, proper allowance for such conditions must be made.

TABLE 4.—RAINFALL AND RUN-OFF FOR PILARCITOS, SAN ANDREAS AND CRYSTAL SPRINGS RESERVOIRS.

Year.	Pilarcitos and San Andreas.		Upper Crystal Springs.		Crystal Springs.	
	Rain, in inches.	Run-off from land only, in inches.	Rain, in inches.	Run-off from land only, in inches.	Rain, in inches.	Run-off from land only, in inches.
1869-70	45.3	12.1	....	....	....	....
1870-71	36.8	6.5	....	....	....	....
71-72	84.6	Uncertain.	....	....	....	....
72-73	41.1	15.0	...	....	....	....
73-74	52.4	19.8	....	....	....	....
74-75	46.6	12.1	....	....	....	....
75-76	72.9	33.8	....	....	....	....
76-77	23.5	2.0	....	....	....	....
77-78	71.6	37.2	51.5	13.8	....	....
78-79	51.9	20.8	37.4	0.0	....	....
79-80	56.5	26.1	40.7	1.1	....	....
1880-81	51.2	29.8	36.9	0.0	....	....
81-82	32.8	14.5	23.6	1.3	....	....
82-83	31.7	12.4	22.9	1.4	....	....
83-84	52.7	25.7	38.0	4.3	....	....
84-85	36.8	14.4	26.5	1.8	....	....
85-86	51.8	30.5	37.3	9.5	....	....
86-87	36.0	17.7	26.0	3.1	....	....
87-88	35.4	14.4	25.4	4.2	....	....
88-89	39.3	14.0	....	....	31.8	11.1
89-90	74.4	37.6	....	....	60.1	44.4
1890-91	37.6	10.2	....	....	30.3	9.5
91-92	37.9	6.9	....	....	30.6	4.2
92-93	54.2	18.1	....	....	43.7	17.8
93-94	49.3	14.6	....	....	39.8	10.0
94-95	64.5	Uncertain.	....	....	52.1	19.0
95-96	47.5	15.1	....	....	38.4	6.3
96-97	48.2	21.1	....	....	39.0	9.2
97-98	24.8	9.5	....	....	20.0	0.0
98-99	40.5	8.9	....	....	32.7	5.1
99-00	42.5	17.2	....	....	38.3	4.7
1900-01	42.1	18.0	....	....	34.0	1.4
01-02	38.9	18.8	....	....	31.5	2.1
02-03	37.5	26.6	....	....	30.3	8.7

The values (above noted) of run-off, for varying amounts of precipitation, have been plotted as ordinates from a base line on which the corresponding amount of rain was scaled off. Thereupon a run-off curve was drawn in the position called for by the individual points with due regard to their weights (determined by the number of years'

records which each represents). But, in the construction of this curve, the fact was taken into account that all water which does not appear in the stream, or, in this case, which does not reach the reservoir, may be considered as having been evaporated, and that, under California conditions, evaporation will be greatest in amount in the long run in the years of greatest rainfall. The more frequent and the greater the soil saturation the greater will be the aggregate evaporation from the soil in the course of the year.

If, now, it were supposed that there was no evaporation whatever, then, in the course of the year, the run-off would equal the rainfall, no matter what the amount of rain, and in such a case the run-off curve would be a straight line starting off from the initial point at an angle of 45°—the scales on which run-off and rain are plotted being the same.

This ideal condition never exists; therefore, for any amount of rain, the corresponding point on the run-off curve must be somewhere below the 45° line, in fact, below by an amount corresponding to the portion of rain which is here credited to evaporation.

If, now, as above stated, the amount of evaporation, expressed in inches over the water-shed, increases with increasing amounts of annual rainfall, the true run-off curve must drop farther and farther below the 45° line as it advances from small to large annual precipitation. For large amounts of rain it will be nearly parallel to the line of complete run-off.

On the other hand, run-off is nil when there is no rain. There may be some run-off due to very small falls of rain, depending upon rain intensity. Consequently, at the initial point, the run-off curve must be tangent to the base line. These considerations were taken into account in the construction of the run-off curves shown in the diagram, Fig. 1.

The stream gauging work of the State Engineer of California, 1878 to 1884, and that of the United States Geological Survey since 1895, have furnished fairly reliable information of river discharge in the central portions of California. This information, combined with an interpretation of rainfall records for the same periods in connection with the State rainfall map, Plate LIX, has made it possible to construct a second run-off curve for areas that are of the high mountainous class (Sierra Nevada with elevations up to 14 000 ft.).

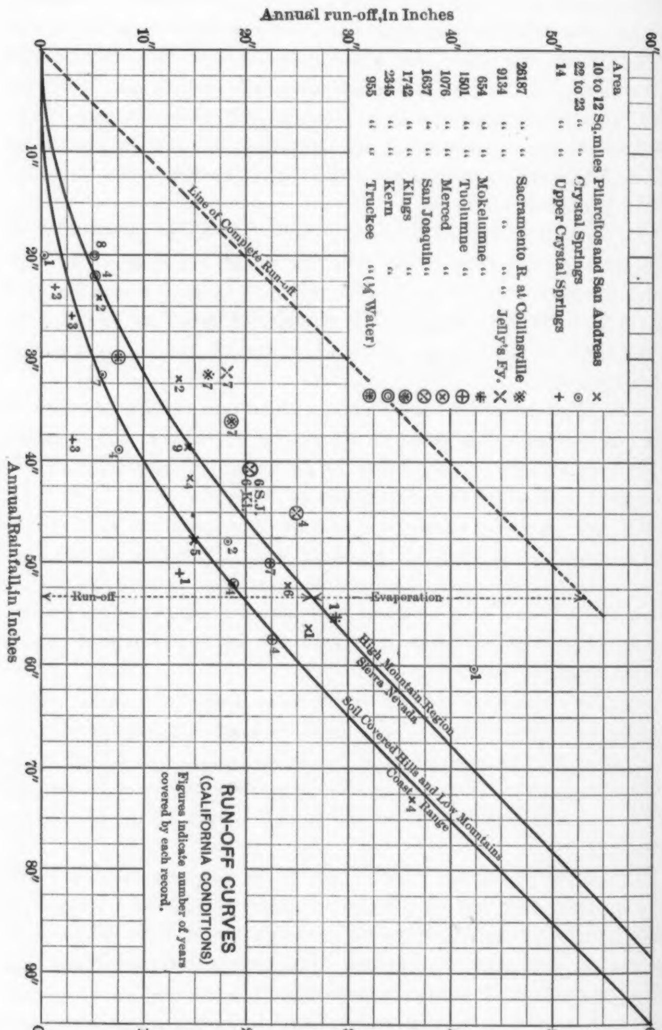

Annual run-off, in Inches
Annual Rainfall, in Inches
Area
10 to 12 Sq. miles Pilarcitos and San Andreas
22 to 23 " " Crystal Springs
14 " " Upper Crystal Springs
26187 " " Sacramento R. at Collinsville
9134 " " " " Jelly's Fy.
654 " " Mokelumne "
1501 " " Tuolumne "
1076 " " Merced "
1637 " " San Joaquin "
1742 " " Kings "
2345 " " Kern "
955 " " Truckee "(M Water)
Line of Complete Run-off
Evaporation
Run-off
High Mountain Region
Sierra Nevada
Soil Covered Hills and Low Mountains
Coast Range
RUN-OFF CURVES
(CALIFORNIA CONDITIONS)
Figures indicate number of years covered by each record.

FIG 1.

Unfortunately, the estimate of precipitation here used, covering large areas, is far from satisfactory. The rainfall stations are too widely scattered. There may be a number of points of rainfall maxima which have not been disclosed by the records. It is believed that in most cases the rain in the large areas has been noted too low. In the large drainage areas, moreover, there is a wide range in the amount of rain falling in different parts of the basin. Thus, in the case of Sacramento River, the normal rain in the basin ranges from 10 in. at some valley points to more than 70 in. in high mountain regions. Moreover, in single rain years, the maximum may be twice the normal. In such years of maximum rainfall there are some portions of such basins as that of Sacramento River from which the run-off is as low as 2 in., and certain other though relatively small portions in which the run-off depth may exceed 100 in. The utilization of an average value for the precipitation, therefore, permits only a crude approximation in such a case to the relation of run-off to rain. It would be much more satisfactory to treat such drainage basins in subdivisions. This could not be done in the case of Sacramento River for lack of data.

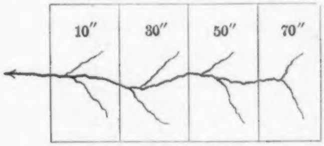


FIG. 2.

The force of the foregoing statement can be made clear by a simple illustration. Suppose a case (Fig. 2) in which the rainfall on one-fourth of a drainage basin of low soil-covered hills is 10 in., on the second fourth, 30 in., on the third fourth, 50 in., and, on the last fourth, 70 in. In this case the run-off, expressed in depth of water as hereinafter shown, will be:

From the first fourth	0.3 in.
" " second "	5.1 "
" " third "	16.9 "
" " fourth "	35.0 "
Average run-off from the entire area	14.3 in.

The average amount of rain on the entire drainage area is 40 in.,

and the run-off for a 40-in. rain is 10 in., or 4.3 in. less than the amount of water which will actually go to the stream.

It may be of interest to record here a rule-of-thumb, for finding the run-off when the rainfall is known, which has never before been published. This rule is:

"The percentage of the annual rainfall, when less than 50 in., which runs to the stream is equal to the number of inches of rain. When the annual rain exceeds 50 in., 25 in. thereof goes to the ground (evaporates), the remainder is run-off."

The reliability of this rule for Pacific Coast rainfall conditions may be judged by comparison with the run-off curves shown on Fig. 1.

Aided by the run-off curves of Fig. 1, the following relation of run-off to rainfall may be noted:

For soil-covered hills and low mountain drainage areas (Coast Range of California, near San Francisco, typical):

For 10	in.	annual	rain,	0.3	in.	run-off.
" 15	"	"	"	0.9	"	"
" 20	"	"	"	1.9	"	"
" 25	"	"	"	3.3	"	"
" 30	"	"	"	5.1	"	"
" 35	"	"	"	7.3	"	"
" 40	"	"	"	10.0	"	"
" 45	"	"	"	13.2	"	"
" 50	"	"	"	16.9	"	"
" 60	"	"	"	25.8	"	"
" 70	"	"	"	35.0	"	"
" 80	"	"	"	45.0	"	"
" 100	"	"	"	65.0	"	"

For drainage basins in high mountain regions (Sierra Nevada Mountains, California, typical):

For 10	in.	annual	rain,	1.0	in.	run-off.
" 20	"	"	"	4.5	"	"
" 30	"	"	"	9.5	"	"
" 40	"	"	"	16.0	"	"
" 50	"	"	"	23.5	"	"
" 60	"	"	"	32.5	"	"
" 70	"	"	"	42.5	"	"
" 80	"	"	"	52.0	"	"
" 100	"	"	"	72.0	"	"

It is not necessary to state that the foregoing run-off figures represent probabilities. They represent mean values for long time-periods. The run-off in any single year may differ from the probable run-off by a wide margin. This possible great variation from the mean is due to the dissimilarity of storm conditions in the successive rainy seasons. Thus, for example, there may be seasons with 25 in. of rain falling in light showers, in which the soil dries out sufficiently after each to be ready to absorb the water of the next. In such a case there would be no run-off. The same amount of rain in one or two severe rain-storms, on the other hand, might produce a run-off several times as great as that indicated as probable by the run-off curve.

The run-off curves shown in Fig. 1 are not in perfect accord with the curves constructed in 1903. It is believed that too great an allowance was then made for water wasted. Consequently, the new curve for low soil-covered areas, based on re-estimates throughout, shows somewhat less run-off for the same quantities of rain than the 1903 curve.*

The run-off curve of 1903 was used in forecasting the probable water yield from a number of water-sheds within the mapped area. In each case the normal annual rain on the drainage basin was determined by reference to the rainfall curves on the map. Thereupon the relation of this normal to the normal at, say Mt. Hamilton, was noted, and from the annuals at Mt. Hamilton the annuals for the drainage basin were computed. These annuals were then used to determine the annual run-off or gross water production of the basin and a mass curve was constructed which served in the ordinary way after storage possibilities had been ascertained to estimate the net annual water yield.

That the net water production of the areas which are tributary to the peninsula reservoirs has been about 500 000 gal. per sq. mile per day has long been known. The gross water production of land only in their water-sheds, according to the figures noted in the preceding tables, has averaged about 20 700 000 gal. per sq. mile per year, or about 570 000 gal. per sq. mile per day. This water production is from an area of about 34 sq. miles on which the normal annual rainfall is about 43 in.

* Municipal Reports of San Francisco, 1901-02. appendix, Water Supply Progress Report; and Municipal Reports of San Francisco, 1903-04, Report on Proposition of Bay Cities Water Company.

DISCUSSION.

Mr. Murphy. E. C. MURPHY, M. AM. SOC. C. E. (by letter).—In this paper the author has given the results of his study of rain distribution over an area of about 5 000 sq. miles lying to the southeast of San Francisco; also, the results of computations of rain and run-off from three small basins in this area. As the rain was measured at 56 places on this area, for periods varying in length from a few years to 54 years, and as the run-off from the three basins was carefully measured for several years, these results are of more than ordinary value.

The normal rain distribution over this area for the 54 years, 1849 to 1903, is shown by isohyetals. Comparing these lines with the lines for this area for the period 1870 to 1901, prepared by the United States Weather Bureau, one finds comparatively little similarity. The number of points where rain is measured is comparatively small, and many of these are in cities at low elevation. Much of the uninhabited country at high elevation is without rain measurement. It is necessary, therefore, in drawing these lines, to supplement the precipitation and elevation data with judgment based on general principles and a personal knowledge of the country. All the rain records do not begin at the same date, and in some of them there are missing months and years. The computed normal at a place depends to some extent on the method used to interpolate the missing data. It is to be expected, therefore, that the location of isohyetal lines by different persons will differ somewhat.

From the isohyetals for each year the mean depth of precipitation over a drainage basin is computed. From them, also, the number of rain gauges necessary in a basin to give a required degree of precision of rain measurement is seen. From them, too, the change in distribution at a given place from year to year is seen.

The author gives a run-off curve for the Coast Range based on his study of the rain and run-off from three small basins. One would naturally expect the data from an area of less than 50 sq. miles, on which the rain and run-off were carefully measured, to be concordant and give a well-defined run-off curve. Such is not true for this case, however. It is seen that all the points representing the data of the Upper Crystal Springs Basin fall to the right of the author's run-off curve, and five of the seven points of the Pilarcitos and San Andreas Basins fall to the left of this curve. It is clear that each of these three basins requires a run-off curve of its own, if the computed run-off is to have a reasonable degree of accuracy.

The third column of Table 5 gives the measured run-off from the Upper Crystal Springs Basin, and the fourth column gives the run-off taken from this curve. The latter, for the 11-year record, is 88% greater than the former.

TABLE 5.—RAIN AND RUN-OFF FROM UPPER CRYSTAL SPRINGS BASIN.

Mr. Murphy.

Year.	Rain, in inches.	Run-off, in inches.	Run-off from curve, in inches.
1877-78	51.5	13.8	18.0
78-79	37.4	0.0	8.5
79-80	40.7	1.1	10.2
80-81	36.9	0.0	8.0
81-82	23.6	1.3	2.6
82-83	22.9	1.4	2.6
83-84	38.0	4.3	8.0
84-85	26.5	1.8	3.6
85-86	37.3	9.5	8.5
86-87	26.0	3.1	3.1
87-88	25.4	4.2	3.0
	Totals.........	40.5	76.1

These data appear to show that run-off curves may differ by a large amount, even for small adjoining basins which appear to be similar.

The author's illustration, Fig. 2, of the necessity of treating drainage basins in subdivisions, in computing the depth of rain, is apt. It shows the need of a large number of precipitation stations in a basin.

The data of this paper show clearly that run-off computed from precipitation is subject to very large errors.

Mr. Duryea.

EDWIN DURYEA, JR., M. AM. SOC. C. E. (by letter).—Mr. Grunsky should be thanked for putting the results of his long investigation of rainfall and stream flow near San Francisco on record for the use of other engineers. During the past five years and more the writer has spent most of his time on similar investigations—studies of the rainfall and stream flow of Coast Range and Sierra areas—with a view to the utilization of the streams for water supply, power, or irrigation. His work has included, not only much study and deduction in the office, both from published records and original records secured in the field, but also the establishment of more than 130 rain gauges and about 20 stream-gauging stations. Therefore, he has accumulated a large number of engineering data on the subject of California rainfall and stream flow, and feels that he can add something to the value of Mr. Grunsky's paper by discussing it. However, as urgent work prevents putting these data into shape for publication at present, the writer is obliged to restrict his discussion of Mr. Grunsky's paper to comments on some statements from which he differs, hoping to present his own rainfall and stream-flow studies for publication somewhat later.

Referring to Mr. Grunsky's high-mountain or Sierra Nevada run-off curve, the writer has checked its correctness quite exhaustively, and finds it to be, as representing a nearly minimum run-off, quite an accurate representation of Sierra conditions. He cannot agree with

Mr. Duryea. Mr. Grunsky, however, that it represents in any way mean values of Sierra run-off. Independently, the writer has worked up the rainfall and run-off relations of the King, Tuolumne, Stanislaus, Mokelumne, Cosumnes, and American Rivers for all rainfall years (September to August, inclusive) previous to, and including, 1906-07, making use of all published and many unpublished data, and of isohyetose lines and numerous base rainfall stations, in order to deduce reasonably accurate average rainfalls for each area gauged and for each year. The results of this study, expressed as points plotted on rectangular co-ordinates, representing rainfall and run-off, respectively, are comprised in 79 points showing the rainfall and its resulting stream flow; and, of the 79 points, only 9 fall appreciably below Mr. Grunsky's curve. Of these 9 points, 5 are for rainfalls of more than 50 in., or for years of such high rainfall that they are of little interest as affecting the reliability of a water supply, and 4 are for rainfalls between 45 and 50 in. For those years in which the rainfall is less than 45 in.—the only years of practical interest in connection with the use of Sierra streams—Mr. Grunsky's curve represents the worst Sierra run-off conditions of which any record has yet been secured, and not mean conditions, as he states.

In plotting his curves with respect to the average rainfalls and stream flows of groups of several years, instead of the rainfalls and stream flows of each separate year (rainfall season), Mr. Grunsky has departed from the usual meaning given to run-off curves by engineers, and, in the writer's opinion, the value and general applicability of his curves have been much lessened by the change. In nearly all investigations for city water supplies (and in a somewhat less degree for power and irrigation projects), the only years of much interest are the two or three forming the period of least run-off or stream flow, generally coincident with the period of least rainfall; and the average water production for several years is not usually of much practical interest. The Spring Valley Water Company's Peninsular system, however, is an exception to this general rule, as the small area furnishing the supply has made it necessary to develop so great a storage that the floods of the best years are utilized by storing the water, even for ten years, until it is needed in the bad years. The Crystal Springs Reservoir, which has been in use nearly 20 years, has been filled but twice, and its dam is as yet uncompleted and still 20 ft. below its proposed ultimate height. It is doubtful if there is any other large system with such excessive storage; and rates of storage and of daily supply per square mile derived from this area are of little value for general application elsewhere. While Mr. Grunsky's method of plotting his Coast Range curve, from means instead of from yearly run-offs, may apply well enough to the fully-developed area from which the data are derived, this could have been achieved as well by a curve

plotted from the actual yearly run-offs, and the data would then have been more safely applicable to similar areas as yet undeveloped. The danger in plotting mean instead of yearly run-offs lies in the minimizing effect thus apparently given to the bad influences of the years of least run-off. To the writer, there seems to be no justification for applying the average run-off method to the Sierra streams, as yet undeveloped. Mr. Duryea.

It is to be regretted that Mr. Grunsky's two run-off curves are shown on one diagram, as the mingling of the symbols relating to the Sierra curve with those relating to the Coast Range curve makes it impossible to study intelligently the relation of each curve to its respective data. The writer has found it necessary to use a separate diagram for each run-off curve projected.

Mr. Grunsky's statement that entering his run-off curve with the average rainfall from a large area of widely varying rainfalls will give too low a run-off is true, theoretically, but the magnitude of the error is dependent on the shape and rainfall characteristics of each area, and is generally very small. Table 6 shows this error for six Sierra rivers, the rainfalls considered being the mean rainfalls as shown by isohyetose lines:

TABLE 6.

River and gauging station.	Area, in square miles.	Run-Offs, by Grunsky's Curve, from: Average rainfall of whole area.	Rainfall of parts between 10-in. isohyetose lines.	Increase in average rainfall, corresponding, by Grunsky's curve, to the larger run-offs.
King River, above Sangers	1 742	8.1 in.	8.6 in. = 106%	3.3%
Tuolumne River, above La Grange	1 557	19.7 "	20.3 " = 103%	1.7%
Stanislaus River, above Knight's Ferry	935	20.2 "	21.0 " = 104%	1.9%
Mokelumne River, above Clements	639	19.2 "	20.1 " = 105%	3.0%
Cosumnes River, above Live Oak Suspension Bridge	580	20.7 "	21.4 " = 103%	1.7%
American River, above Fair Oaks	1 925	26.4 "	27.2 " = 103%	1.5%

The mean rainfalls on the King River area vary from about 9 in. in the valley to 44 in. in the high mountains, those of the Tuolumne River from about 16 to 57 in., the Stanislaus from 18 to 58 in., the Mokelumne from 20 to 60 in., the Cosumnes from 26 to perhaps 63 in., and the American from about 25 to perhaps 65 in.; yet the error in run-off from entering the Grunsky curve with the average mean rainfall of each entire area is only from 3 to 6%, with an average error of only 4% for the six streams. If the average mean rainfalls of the entire areas are increased by from 1.5 to 3.3%, with an average increase of 2.2%, the run-offs obtained from the curve will then equal

Mr. Duryea. those resulting from dividing up each area into from five to nine parts and getting the run-off of each part separately. Because of the small error shown for these six streams, and also because all but minimum run-offs are represented as well by percentages of the rainfall as by curves, it seems that only in very unusual instances will the treatment of large areas as a whole cause errors in the resulting run-off of any importance. It is probable that the streams rising on the east slopes of the Sierra Nevadas and flowing into the Great Basin are exceptions to this general rule.

Messrs. Haehl and Toll. H. L. HAEHL, Assoc. M. Am. Soc. C. E., and ASAHEL C. TOLL, Jun. Am. Soc. C. E. (by letter).—The difficulty of determining the mean rainfall for any considerable area in the mountain water-sheds of California is well recognized by hydraulic engineers having to deal with water-supply or power development within the State. This difficulty, due in large measure to the great variations in rainfall conditions within even very small areas, is accentuated by the fact that the longest and most reliable rainfall records are those from valley stations, so that all local records have to be amended by comparison with these longer records before reliable mean seasonal rainfalls can be deduced for the mountain stations, and even then it remains to decide how much territory is represented by each station.

A valuable aid in this determination is an isohyetose or rainfall distribution map such as that presented by the author. Such a map, based on all available data, presents in graphic form all known facts of the distribution of rainfall over the area covered, and affords a ready means of judging the mean seasonal rainfall at any point or for any included area. The writers, having access to many data not available to the author at the time of his investigation, have worked out, for the territory covered by those data, a revised isohyetose map; and they present herewith the results of their studies as an additional contribution to the knowledge upon this subject now made available to the profession.

The records here used are largely the results of recommendations made in 1902 by F. S. Washburn, M. Am. Soc. C. E., then Consulting Engineer to the Bay Cities Water Company, the observations having been made within the catchment areas of that company. The collection of these and other data, covering evaporation, humidity, and stream flow, in Santa Clara County, as well as the other engineering activities of that company, was under the direction of Edwin Duryea, Jr., M. Am. Soc. C. E., who became, and for several years continued to be, its Chief Engineer. The field work was under the direct supervision of H. L. Haehl and K. F. Cooper, Associate Members, Am. Soc. C. E., as Assistant Engineers. Unfortunately, most of the final tabulations of these data, together with the various studies in which they were used, were burned in the San Francisco offices of the company

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during the Earthquake-fire of April, 1906, and the work of collecting and re-compiling the large mass of information, preserved in the field offices in the original field books and notes, has not since been undertaken in full. It is hoped that the records covering run-off, evaporation, etc., in Santa Clara County, which are unusually complete and extensive, may at some time form the basis for a paper before this Society, but, owing to the lack of time for putting those data into presentable form, the present discussion will be confined principally to the question of rainfall distribution.

The collection of rainfall records was begun in December, 1902, when 24 rain gauges were located at various representative points in the 193 sq. miles of catchment area of the Coyote River. In September, 1903, at the beginning of the rainfall season, 1903-04, 24 additional rain gauges were set on the Coyote catchment area; 22 on the Uvas and Llagas Creeks, representing 53 sq. miles of catchment area; and 34 on the Arroyo Honda and Arroyo Valle catchment areas, of 88 sq. miles. Later, several additional rain gauges were located at various points in these areas, bringing the number of rainfall stations maintained by the company up to more than 100; and with the stations maintained by the U. S. Weather Bureau, the Southern Pacific Railway, Lick Observatory, and a few individuals, making a total of 122 record stations within the area of 1 000 sq. miles shown on Plate LX, or about 1 to each 8 sq. miles. Most of the company's records were discontinued in 1905, but some have been brought up to date and will probably be continued indefinitely. The periods of record of the various gauges are given in Tables 8 to 13, which deal with their records and the deductions therefrom, treating them in groups according to their location and the similarity of their rainfall characteristics to those of certain nearby stations, the longer-period records of which were available for use in this study.

The position and extent of each group and the relation of the entire area here considered to the larger area covered by the author's map are shown by Fig. 3, which shows also the relative location of the stations, the longer-period records of which were used in deducing mean seasonal rainfalls for the stations with short-record periods. Upon Plate LX are shown the location of each rain gauge, with its deduced mean seasonal rainfall, and the isohyetose lines based on these data. The author's isohyetose lines for this area are copied from his map to Plate LX, thus presenting a ready means of comparison between the author's deductions, based on data available in 1903, and the writers' deductions, based on later and more complete data now available.

Because most of the area covered by these records was uninhabited, it was not possible to obtain daily rainfall readings for many of the gauges. Daily records were procured, as far as possible, however, by

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locating gauges where they could be observed by field parties engaged in stream gauging, surveying, or construction, and by those few residents whose interest could be sufficiently aroused. In this way ten daily records were secured, in addition to those obtained from other observers, such as the Weather Bureau, the Southern Pacific Railway, etc.

Owing to the roughness of the country and the lack of wagon roads, together with the number of gauges to be observed and the distances between them, four mounted observers were constantly employed in reading gauges, with a fifth assistant for part of the time. In this way, under ordinary conditions, each gauge was visited at intervals of

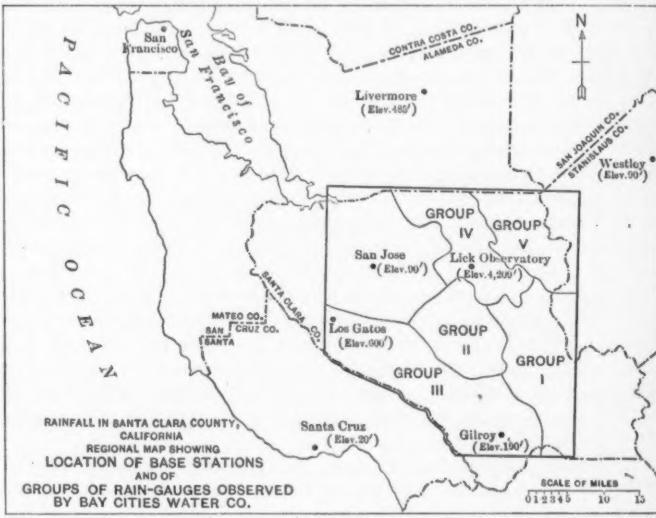
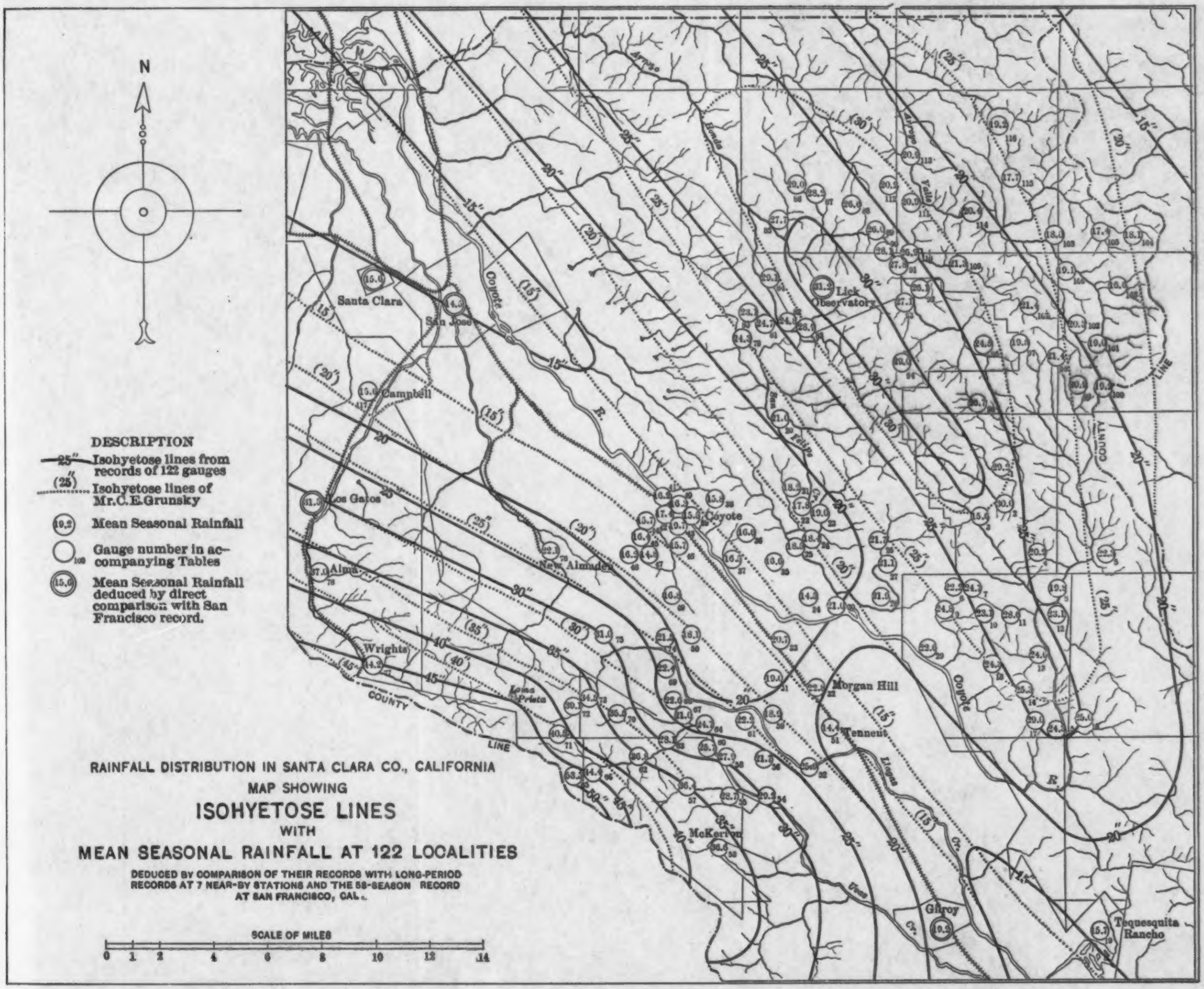


FIG. 3.

about one week. By special effort, during periods of unusual rainfall, it was possible to reduce the interval to 4 or 5 days for most of the gauges, while some few could not be reached for periods of several weeks during long-sustained floods, as certain streams were then impassable. Such gauges were provided with large overflow tanks, and in this way complete records were obtained.

The rain gauges were of the standard type in use by the U. S. Weather Bureau, except that they were somewhat smaller, being 3 in. in diameter. It is probable that the records are generally slightly below the true rainfall, owing to the evaporation from the gauge during the intervals between readings. This fact is best seen by com-



RAINFALL DISTRIBUTION IN SANTA CLARA CO., CALIFORNIA
MAP SHOWING
ISOHYETOSE LINES
WITH
MEAN SEASONAL RAINFALL AT 122 LOCALITIES
DEDUCED BY COMPARISON OF THEIR RECORDS WITH LONG-PERIOD RECORDS AT 7 NEAR-BY STATIONS AND THE 58-SEASON RECORD AT SAN FRANCISCO, CAL.

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paring the records of Gauges 42 and 43 at Coyote, which were read daily, with the slightly lower records from the large number of gauges surrounding them, read weekly. The condition was suggested by records elsewhere, but the evidence is most positive in this locality, due to the many rainfall records observed in conjunction with evaporation records in this neighborhood. It has not been considered necessary, however, to attempt a correction of the records for loss by evaporation, because the error introduced is slight.

The main topographic feature of the region here considered is the Santa Clara Valley, a broad, fertile plain extending northeast and southwest midway of the area. Beginning at San Francisco Bay, this valley is about 15 miles in width; southeasterly therefrom it gradually narrows and rises, reaching its least width of about ½ mile at Coyote, and its maximum elevation of about 350 ft. near Morgan Hill. Beyond Coyote it widens again gradually to 5 miles at Gilroy, while the elevation decreases to sea level at Monterey Bay. The influence of these changes in elevation and width upon the rainfall is clearly shown on Plate LX.

The greater precipitation shown by the records at Coyote and Morgan Hill, which the author has ignored in the construction of his isohyetose lines, is not a freak of these particular gauges, but the logical result of existing conditions, as the numerous additional records here presented clearly demonstrate. Between the Santa Clara Valley and the Pacific Ocean to the westward lies the Santa Cruz Range, the crest of which, within the area under consideration, corresponds closely to the county line shown on Plate LX as part of the boundary of the area. This range, reaching an elevation of about 3 800 ft., is the first to intercept southwest storms from the ocean, and the rainfall upon it is the greatest within the area. Lying to the eastward of the Santa Clara Valley, and completing the area here considered, is the Mount Hamilton Range, which reaches a maximum elevation of more than 4 000 ft. This range consists of a series of ridges, having a northwest-southeast trend, separated by fault valleys or cañons paralleling the Santa Clara Valley, which also occupies the basin along a fault zone.

As will be noted by reference to Plate LX, the rainfall upon this range is much less than upon the one to the westward. This rainfall comes for the most part with southeast winds which follow the axis of the valley from Monterey Bay. Of the moisture carried eastward from the sea the greater portion is intercepted by the Santa Cruz Range, the remainder being precipitated chiefly at the higher elevations in the Mount Hamilton Range.

The rainfall throughout this territory, as the author has pointed out, occurs chiefly during seven months of each season (November to May), more than half usually coming during December, January, and February. There is seldom any precipitation during June, July, and

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August. A light fall of snow, which melts rapidly, usually occurs during each season at the higher elevations in the Mount Hamilton Range, but only occasionally does snow fall in the Santa Cruz Range, and very seldom has any fallen in the foot-hills of either range, or in the valley. As a consequence of the uneven distribution of rainfall through the season, and of the comparatively steep gradients of the mountain stream beds, the greater part of each season's run-off occurs in the winter.

The two mountain ranges differ radically in their run-off characteristics, due no doubt to the difference in the forestation of the two areas. The Santa Cruz Range, except at its highest elevations, where chaparral predominates, sustains an abundant growth of timber, supported, probably, by its very heavy rainfall. In consequence, the flow of its streams responds less rapidly to heavy rainstorms, and remains well sustained through the dry season of each year. The extent of forestation in the Mount Hamilton Range is very much less, for, while the cañon bottoms and protected slopes are well covered with trees and underbrush, a considerable portion of the exposed upland sustains only chaparral or grass. It follows, naturally, that the greater part of the season's stream flow in this range occurs in torrents which rise rapidly after a heavy rainfall, reach their maximum flow within a few hours, and entirely recede within a few days; and also that the summer flow is almost negligible. The two ranges differ greatly, also, in the volume of run-off from like areas each year, due probably to the considerable difference in rainfall intensity in the two districts.

In deducing the mean seasonal rainfalls presented on Plate LX, the local records were compared with the 58-season record at San Francisco, that being the only nearby record of more than 35 seasons' length, and the longest record in this part of the State. For many of the rain gauges, owing to their short period of record, it was necessary to make this comparison indirectly, comparing the gauge records directly with those at nearby stations, and through the medium of their longer-period records a better basis of comparison was obtained. To determine what records to use in this intermediate capacity, a preliminary study was made of the characteristics of rainfall throughout the area. This study involved a comparison of the rainfall at various rain gauges in all parts of the area with the rainfall at the possible base stations, including the comparison of the records for each of the heavier storms as well as of the seasonal totals. The purpose, here, of course, was to determine, not what base station or combination of stations had the same quantity of rainfall as the rain gauges involved, but rather what stations were affected by the same storms and had records showing an approximately constant ratio to the gauge record. This study gave a very good basis for the selection of those base stations which would afford the most accurate means of deduction.

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From this preliminary study and the writers' personal knowledge of the distribution of storms and the physical conditions influencing rainfall within the region, the rain gauges were divided into groups as follows:

Group I.—Rain gauges on the upper reaches of the Coyote River, in mountains ranging from 1 200 to 2 800 ft. in elevation, the records for which corresponded very closely in their rainfall distribution with the mean between Gilroy and Lick Observatory records.

Group II.—Rain gauges on the lower reaches of the Coyote River, on the San Felipe catchment area, and in the Santa Clara Valley, ranging in elevation from 1 900 down to 240 ft., the records of which show the same laws of distribution as the mean between the San José and Gilroy records.

Group III.—Rain gauges in the Santa Cruz Mountains between the Santa Clara Valley and the summit in the neighborhood of Loma Prieta, including the catchment areas of the Uvas and Llagas Creeks, with elevations varying from 300 to 3 800 ft. Rainfalls recorded by these gauges correspond at times to the mean of the Lick Observatory and Los Gatos records, at other times to the mean of the Gilroy, Santa Cruz, and Los Gatos records, while, for the period of record at the majority of the gauges of the group, the mean of records at all four stations (Lick Observatory, Gilroy, Santa Cruz, and Los Gatos) gives a very reliable basis of deduction.

Group IV.—Rain gauges in the Arroyo Honda catchment area, near Mount Hamilton, at elevations from 2 200 to 3 600 ft., and on the headwaters of San Felipe Creek at 1 400 and 2 200 ft. elevation, the records of which correspond in their seasonal fluctuations with the mean of the Gilroy, Lick Observatory, and Livermore records.

Group V.—Gauges in the Arroyo Valle catchment area, at elevations varying from 1 800 to 2 600 ft., for which the best basis of deduction was found to be the mean of the Gilroy, San José, Livermore, Westley, and Lick Observatory records.

The location of the areas containing the groups of gauges just cited is shown on the map, Fig. 3, together with the positions and elevations of the various base stations selected. The conclusions as to similarity in rainfall variations between the gauges of each group and the base stations chosen in each case will be seen to find further general justification from their relative locations on this map.

Having determined what stations would be used as intermediate base stations, their 58-season mean rainfall was deduced by comparing their mean seasonal rainfall for their period of record with the mean seasonal rainfall at San Francisco for the same period, and assuming that the relation between these means also existed between the 58-season means. No single record of San Francisco rainfall could be used for this comparison, the only available record extending back to

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1849 (that published by the U. S. Weather Bureau) being a composite made up of records kept successively at six different points in San Francisco. The first four of these places, having records covering the period from 1849 to 1892, had sufficiently similar rainfall conditions to warrant their being considered as one station; but the last two, with records covering the period, 1892-1907, being on the roofs of high buildings, had conditions so dissimilar to those of the first four that the use of their records in the same series with the earlier records would give an erroneous mean seasonal rainfall for San Francisco and a false basis for the deduction of the mean seasonal rainfall at other stations, the shorter-period records of which are lengthened by comparison with the San Francisco record, and means thereby determined.

It is to be regretted that in a publication of the character of Bulletin L* of the U. S. Weather Bureau no mention is made of the sources of, and the conditions affecting, the reliability of the San Francisco record there published, especially in view of the very noticeable and well-recognized difference between records taken at or near the ground, and those taken at considerable elevation above the ground in the same locality. The author has called attention to the fact that the Weather Bureau record as published could not be used safely, and indicated his method of avoiding error due to its use. It is unfortunate that he was not able to publish the composite record used by him, together with those records which were the basis for it. In recent correspondence with the writers he pointed out further the break in the Weather Bureau record just referred to, and cited in full the various records used by him in forming his composite record, though he was unable to supply a copy of any of these. An attempt to collect these records developed the fact that some of them were destroyed in the Earthquake-fire of 1906.

The writers, therefore, have deduced a continuous, comparable San Francisco record on a different basis from that used by the author. The record sought was not one which should necessarily show the true mean rainfall for San Francisco, but one which would be strictly comparable to outside records; in other words, a record in which each season should bear the correct ratio to the mean; such ratio, and not the absolute record, being of importance as the basis for lengthening outside shorter-period records to the 58-season period, and obtaining comparable long-time means. As has been previously pointed out, the published Weather Bureau record falls short of this requirement by reason of the changed conditions governing it after 1892. However, careful rainfall records were kept in San Francisco by Mr. John Pettee, under favorable and unchanging conditions, covering 27 seasons prior to 1892 and 9 seasons thereafter; and these records form a good basis for the correction of the Weather Bureau record. Fortunately, these Pettee records are available in permanent form, having been published

* "Climatology of California."

in Bulletin L, U. S. Weather Bureau. As will be seen by reference to Table 7, Column 4, the Pettee records bore a reasonably constant relation to the published Weather Bureau records prior to 1892, which relation for the entire period, 1865 to 1892, was 97.2 per cent. Immediately following the removal of the Weather Bureau gauge to the roof of the Mills Building, in 1892, this relation increased suddenly, and, for the 9 seasons following, averaged 134.4 per cent. The change of relation is, of course, a measure of the effect of change of elevation and location upon the Weather Bureau record, the Pettee record being consistent throughout. Messrs. Haehl and Toll.

To obtain a consistent 58-season record for San Francisco, accepting the Weather Bureau rainfall as published up to 1892, a record from 1892 to 1907 was deduced, therefore, which as a whole bears the same relation to the Pettee records as was borne during the 27-year period prior thereto, the rainfall varying in the individual seasons with the observed Weather Bureau records. The resulting record made use of in the deductions of means elsewhere is given in Table 7, together with the full data used and the intermediate steps taken.

The rainfalls shown in Column 2 of Table 7 are taken from an advance sheet published by the San Francisco office of the U. S. Weather Bureau in 1908, as the San Francisco rainfall. This record is a composite, stated by Professor A. G. McAdie, under whose direction it was published, to be the record kept by Mr. Thomas Tennent, maker of nautical instruments, from 1849 to 1871, and the U. S. Signal Service and U. S. Weather Bureau records thereafter. These records were obtained under similar conditions from 1849 to 1892. The records from 1892 to 1907, however, form a series not comparable with the records for the earlier period.

The rainfalls shown in Column 3 are taken from Bulletin L, U. S. Weather Bureau, and represent a record of rainfall in San Francisco, carefully kept by Mr. John Pettee at a single gauge, near the ground, under unchanging conditions.

The percentages in Column 4, showing the relations between seasonal rainfalls in the continuous and the discontinuous record, clearly illustrate the break of 1892 in the Weather Bureau record, and indicate means of deducing a continuous record for the entire period from 1849 to 1907.

Column 5 shows the Pettee record lengthened to cover the 58 seasons by the use of two assumptions: (1) That the Pettee record would have shown the same relation to the U. S. Weather Bureau record from 1849 to 1865 which it did show for the period, 1865-1892; (2) That for the seasons, 1901-02 to 1906-07, the Pettee record bore to the U. S. Weather Bureau record the relation shown for the period 1892-1901. (See Column 4.)

Column 6 shows the deduced San Francisco record. This is the

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TABLE 7.—DEDUCTION OF A CONTINUOUS, COMPARABLE, RAINFALL RECORD FOR SAN FRANCISCO, CAL.

(1)	(2)	(3)	(4)	(5)	(6)
Season or period.	RECORDED RAINFALL, IN INCHES. United States Weather Bureau. Season.* Total.	RECORDED RAINFALL, IN INCHES. Pettee. Season.* Total.	Percentage relation: Pettee / U. S. W. B.	DEDUCED RAINFALL, IN INCHES. Pettee. Season.* Total.	DEDUCED RAINFALL, IN INCHES. San Francisco. Season.* Total
1849-50	33.10	......	......	Not deduced by seasons.	33.10 Same as U. S. W. B. record.
50-51	7.44	......	......		7.44
51-52	18.44	......	......		18.44
52-53	35.30	......	......		35.30
53-54	23.84	......	......		23.84
54-55	23.75	......	......		23.75
55-56	21.68	......	......		21.68
56-57	19.94	......	......		19.94
57-58	21.97	......	......		21.97
58-59	22.03	......	......		22.03
59-60	22.46	......	......		22.46
60-61	19.51	......	......		19.51
61-62	49.27	......	......		49.27
62-63	13.74	......	......		13.74
63-64	10.29	......	......		10.29
64-65	24.52	......	......		24.52
1849-65	367.28		(assumed) 97.2	357.07	367.28
1865-66	22.93	23.57	102.7	Same as Pettee recorded rainfall.	22.93 Same as U. S. W. B. recorded rainfall.
66-67	34.92	35.94	102.9		34.92
67-68	38.84	40.62	104.5		38.84
68-69	21.35	20.66	96.7		21.35
69-70	19.31	20.12	104.2		19.31
70-71	14.13	13.12	92.9		14.13
71-72	30.77	28.89	93.9		30.77
72-73	15.74	19.59	124.4		15.74
73-74	24.64	24.50	99.4		24.64
74-75	20.56	18.15	88.3		20.56
75-76	31.21	28.31	90.7		31.21
76-77	11.04	9.98	90.4		11.04
77-78	35.17	32.80	93.2		35.17
78-79	24.46	22.14	90.5		24.46
79-80	26.63	23.61	88.6		26.63
80-81	29.86	27.24	91.2		29.86
81-82	16.14	15.83	98.0		16.14
82-83	20.12	19.59	97.3		20.12
83-84	32.42	29.18	90.0		32.42
84-85	18.12	17.07	94.2		18.12
85-86	31.50	28.75	91.2		31.50
86-87	18.82	16.68	88.6		18.82
87-88	16.75	16.95	101.1		16.75
88-89	23.85	24.25	101.6		23.85
89-90	45.86	46.43	101.2		45.86
90-91	17.68	18.74	106.5		17.68
91-92	18.41	20.13	108.9		18.41
1865-92	661.23	642.84	97.2	642.84	661.23

* The season here used is from September to August, inclusive.

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TABLE 7.—(*Continued.*)

(1)	(2)		(3)		(4)	(5)		(6)	
	Recorded Rainfall, in Inches.					Deduced Rainfall, in Inches.			
Season or period.	United States Weather Bureau.		Pettee.		Percentage relation: Pettee / U. S. W. B.	Pettee.		San Francisco.	
	Season.*	Total.	Season.*	Total.		Season.*	Total.	Season.*	Total.
1892-93	21.77		27.15		124.7	Same as Pettee recorded rainfall.		30.09	= U. S. W. B. × 134.4% ÷ 97.2%.
93-94	18.45		24.44		132.5			25.51	
94-95	25.71		34.94		135.8			35.55	
95-96	21.37		29.45		137.8			29.54	
96-97	23.30		30.91		132.6			32.21	
97-98	9.38		13.83		147.4			12.97	
98-99	16.87		23.96		142.0			23.32	
99-00	18.47		25.40		137.5			25.53	
00-01	21.17		27.11		128.1			29.25	
1892-1901		176.49		237.19	134.4		237.19		
1901-02	18.98		......		......	25.50		26.24	
02-03	18.28		......		......	24.58		25.27	
03-04	20.67		......		......	27.78		28.58	
04-05	23.37		......		......	31.41		32.31	
05-06	20.61		......		......	27.71		28.49	
06-07	25.98		......		......	34.90		35.91	
1901-07		127.89		......	(assumed) 134.4		171.88		
1892-1907		304.38					409.07		420.77
1849-1907		1 332.89		......	......		1 408.98		1 449.28
58-Season Mean		22.98		......	......		24.29		24.99

* The season here used is from September to August, inclusive.

same as the U. S. Weather Bureau record in Column 2 from 1849 to 1892. The rainfall from 1892 to 1907 is deduced on the assumption that the deduced Pettee rainfall in Column 5 bears the same relation to the deduced San Francisco rainfall for the period 1892-1907 as it bears for the period, 1849-1892 (97.2%). Since this deduced Pettee rainfall is 134.4% of the U. S. Weather Bureau recorded rainfall for the later period, it follows that the relation between the deduced continuous San Francisco rainfall and the recorded U. S. Weather Bureau rainfall for the period, 1892-1907, is 134.4 ÷ 97.2 = 138.3 per cent. This percentage relation is applied to each season's record from 1892-93 to 1906-07, given in Column 2, to obtain that given in Column 6.

As understood from Mr. Grunsky's paper, the San Francisco record used by him was an average of all available records for each season, the number of records varying from season to season, in some cases being but one and in others as many as five. It is believed that, for the end

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sought, the method followed by the writers is more logical, though possibly less representative of the mean rainfall in San Francisco.

The mean seasonal rainfall for each locality, as given on Plate LX, was deduced, in the same manner as for base stations, by comparing the total recorded rainfall at the gauge with the total for the same period shown by the mean of the base stations used. It was thought best not to determine and compare short-period means, but, instead, to make the comparisons by totals in this case, since two sets of fractional seasons were used, namely: (1) January to August, 1903, inclusive, for 22 gauges having readings which were begun in December, 1902, and (2) September, 1907, to March, 1908, inclusive, for 5 gauges having records which were available up to the latter date. The stations having long-period records which were here used as the basis of deduction are given in Table 8, together with the period and length of each record and the steps taken in deducing the 58-season mean from the record at San Francisco. In Tables 9 to 13 are given the records for gauges in Groups I to V, respectively. These tables contain also the elevations of each gauge (obtained in most cases by repeated aneroid readings and hence fairly reliable), the number of seasons of record, and the steps taken in deducing each long-period mean, as has just been explained.

TABLE 8.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS. BASE STATIONS.

Using San Francisco Record as Basis.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Station.	Period of record.	Number of seasons.	Mean Seasonal Rainfall for Period of Station Record, in Inches.		Percentage relation: Col. 4 / Col. 5.	Deduced 58-season mean at station, in inches.
			At station.	At San Francisco.		
San Francisco....	'49-50 to '06-07	58	24.99	24.99	100.0%	24.99
Santa Clara....	'81-82 " '84-85 '87-88 " '06-07	24	16.33	26.25	62.2	15.55
San José..........	'74-75 " '06-07	33	15.12	26.10	57.9	14.47
Los Gatos.........	'85-86 " '06-07	22	34.09	27.06	125.9	31.46
Santa Cruz.......	'78-79 " '06-07	29	27.28	26.31	103.6	25.90
Gilroy............	'74-75 " '06-07	33	20.10	26.10	77.0	19.24
Lick Observatory	'81-82 " '06-07	26	32.77	26.23	124.8	31.20
Livermore......	'71-72 " '00-01 '02-03 " '06-07	35	15.76	25.89	60.8	15.20
Westley...........	'89-90 " '06-07	18	10.74	27.93	38.4	9.60

Having located all the available rainfall record stations on Plate LX, the isohyetose lines were drawn in the manner described by the author, giving due weight to each record in accordance with its length and the influence of its known location upon its value as a representa-

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tive record. It may be noted that in very few instances are the local records at great variance with the isohyetose lines as drawn. The most notable instance of such disagreement is Gauge No. 3 on the Upper Coyote, surrounded by gauges showing means varying from 20 to 30 in., and yet giving by its record only 15.6 in. This difference is not due to any misreading of the gauge, for the record consistently bears about the same ratio to records of neighboring gauges for every storm, and no reason is apparent for its disagreement. Other examples of disagreement are Gauge No. 34 and Gauge No. 51, the latter located at Tennent. The Tennent gauge has been given considerable weight on account of its longer record, though it is seemingly much too low, and nothing is known to the writers concerning the care with which it was located or observed.

A study of the isohyetose lines on Plate LX will show how greatly the topography affects the distribution of rainfall. The effect of the gradual rise in the valley floor toward Morgan Hill has been referred to, and attention is called to the variations in the lines of equal rainfall caused by the ridge extending northeasterly from Loma Prieta, the variation being more accentuated by the influence of the Uvas

TABLE 9.—Long-Period Mean Seasonal Rainfall Deductions. Group I; Gauges 1-19.

Using Mean of Gilroy and Lick Observatory Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with approximate elevation).	Period of record.		Total Recorded Rainfall for Period of Gauge Record, in Inches. At gauge.	Mean of base stations.	Relation: $\frac{\text{Col. 3}}{\text{Col. 4}}$	58-Season Mean, in Inches. For base stations.	For gauge (deduced).
1 (1870)	Jan. '03-Aug. '05	2⅔ Seasons.	80.46	69.44	115.8%	25.22	29.2
2 (1800)	" "	2⅔ "	82.69	......	119.0	......	30.0
3 (2730)	" "	2⅔ "	43.02	......	61.9	......	15.6
4	" "	2⅔ "	55.50	......	79.9	......	20.2
5 (2440)	" "	2⅔ "	61.51	......	88.6	......	22.3
6 (2500)	" "	2⅔ "	61.23	......	88.2	......	22.2
7 (2800)	" "	2⅔ "	67.93	......	97.8	......	24.7
8 (1350)	" "	2⅔ "	54.40	......	78.3	......	19.8
9 (1720)	" "	2⅔ "	68.26	......	98.3	......	24.8
10 (2230)	" "	2⅔ "	63.60	69.44	91.6	......	23.1
11 (1490)	Sep. '03-Aug. '06	3 "	97.26	85.84	113.3	......	28.6
12 (1240)	Sep. '03-Aug. '05	2 "	47.59	51.92	91.6	......	23.1
13 (1220)	Jan. '03- "	2⅔ "	67.69	69.44	97.5	......	24.6
14 (2720)	" "	2⅔ "	69.71	......	100.4	......	25.3
15 (1210)	" "	2⅔ "	66.82	......	96.2	......	24.3
16 (1200)	" "	2⅔ "	68.82	69.44	99.1	......	25.0
17	Sep. '03- "	2 "	59.58	51.92	114.8	......	29.0
18 (2780)	" "	2 "	50.01	51.92	96.3	......	24.3
19	Sep. '99--Aug. '06	7 "	113.48	182.02	62.3	25.22	15.7

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TABLE 10.—Long-Period Mean Seasonal Rainfall Deductions.

Group II; Gauges 20-51.

Using Mean of Gilroy and San José Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with approximate elevation).	Period of record.		Total Recorded Rainfall for Period of Gauge Record, in Inches.		Relation: Col. 3 / Col. 4	58-Season Mean, in Inches.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
20 (1050)	Jan. '03-Aug. '05	2½ Seasons.	57.63	46.36	124.4%	16.86	21.0
21	Sep. '03- "	2 "	37.67	34.97	107.6	......	18.2
22	" "	2 "	37.01	34.97	105.9	......	17.8
23 (750)	Jan. '03- "	2½ "	52.16	46.36	112.5	......	19.0
24 (650)	" "	2½ "	50.44	46.36	108.9	......	18.4
25 (640)	Sep. '03- "	2 "	38.36	34.97	109.6	......	18.5
26 (1250)	Jan. '03- "	2½ "	59.57	46.36	128.4	......	21.7
27 (1300)	" "	2½ "	57.85	46.36	124.8	......	21.1
28 (1560)	" -Aug. '06	3½ "	89.26	68.62	130.0	......	21.9
29 (1870)	Sep. '03-Aug. '05	2 "	46.78	34.97	133.8	......	22.6
30 (440)	" -Mar. '08	5 "	118.35	95.11	124.4	......	21.0
31	" -Aug. '05	2 "	39.43	34.97	112.8	......	19.0
32 (350)	Sep. '99-Aug. '04	5 "	135.67	100.36	135.2	......	22.8
33 (345)	Jan. '03-Mar. '08	5½ "	130.51	106.50	122.5	......	20.7
34	Sep. '03-Aug. '05	2 "	30.04	34.97	85.9	......	14.5
35 (1175)	" "	2 "	34.45	......	98.5	......	16.6
36	" "	2 "	34.44	34.97	98.5	......	16.6
37 (310)	" -Aug. '07	4 "	81.47	83.09	98.0	......	16.5
38	" -Aug. '05	2 "	32.79	34.97	93.7	......	15.8
39 (248)	" -Aug. '07	4 "	76.56	83.09	92.2	......	15.5
40 (246)	Jan. '03-Aug. '05	2½ "	44.94	46.36	96.9	......	16.3
41 (242)	Sep '03- "	2 "	33.74	34.97	96.4	......	16.2
42 (248)	" -Mar. '08	5 "	98.37	95.11	103.4	......	17.4
43 (255)	Sep. '99-Aug. '04	5 "	117.32	100.36	116.9	......	19.7
44 (310)	Sep. '03-Aug. '07	4 "	77.50	83.09	93.3	......	15.7
45 (250)	" "	4 "	80.61	83.09	97.0	......	16.4
46 (300)	" -Aug. '06	3 "	53.37	57.24	93.3	......	15.7
47	Sep. '04-Aug '05	1 "	18.01	20.48	87.9	......	14.8
48	" "	1 "	19.64	......	96.0	......	16.2
49	" "	1 "	20.35	......	99.4	......	16.8
50	" "	1 "	21.96	20.48	107.2	......	18.1
51 (325)	Sep. '78-Aug. '94	16 "	239.04	280.33	85.3	16.86	14.4
117 *(195)	Sep. '97-Aug. '04	7 "	97.34	*142.98	68.0	*22.96	15.6

*Also Gauge 117, using mean of San José and Los Gatos records as basis.

Creek Cañon, paralleling the ridge to the south. It may be noted, further, that the series of fault cañons in the Mount Hamilton Range have decidedly influenced the distribution of rainfall in that region. These effects of the topography are not only shown by the records here presented, but are known to the inhabitants of the region, and have been recognized by the writers from general observations extending through several years.

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TABLE 11.—Long-Period Mean Seasonal Rainfall Deductions. Group III; Gauges 52-78.

Using Mean of Gilroy, Lick Observatory, Santa Cruz, and Los Gatos Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with approximate elevation).	Period of record.		Total Recorded Rainfall for Period of Gauge Record, in Inches.		Relation: Col. 3 / Col. 4	58-Season Mean, in Inches.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
52 (360)	Sep. '06-Mar. '08	2 Seasons	55.25	57.45	96.2%	26.95	25.9
53	Sep. '92-Aug. '02	10 "	377.40	277.64	135.9	......	36.6
54 (535)	Sep. '03-Aug. '05 Sep. '06-Mar. '08	4 "	125.45	115.77	108.4	......	29.2
55 (360)	Sep. '03-Aug. '05	2 "	62.18	58.32	106.6	......	28.7
56 (640)	" "	2 "	47.05	......	80.7	......	21.7
57 (470)	" "	2 "	78.67	......	135.0	......	36.4
58 (460)	" "	2 "	60.40	......	103.6	......	27.9
59 (580)	" "	2 "	40.91	......	70.1	......	18.9
60 (340)	" "	2 "	55.72	......	95.5	......	25.7
61 (690)	" "	2 "	47.92	......	82.2	......	22.2
62 (845)	" "	2 "	79.55	......	136.5	......	36.8
63 (880)	" "	2 "	60.76	......	104.2	......	28.1
64 (620)	" "	2 "	53.55	......	91.8	......	24.7
65 (1 900)	" "	2 "	115.75	......	198.5	......	53.5
66 (965)	" "	2 "	95.50	......	163.8	......	44.2
67 (660)	" "	2 "	45.51	......	78.0	......	21.0
68 (732)	" "	2 "	48.90	......	83.9	......	22.6
69 (730)	" "	2 "	48.51	......	83.2	......	22.4
70 (930)	" "	2 "	76.73	......	131.6	......	35.5
71	" "	2 "	87.54	......	150.2	......	40.5
72 (1 190)	" "	2 "	84.63	58.32	145.2	......	39.1
73	" -Aug. '04	1 "	35.41	27.70	127.8	......	34.5
74 (745)	" -Aug. '05	2 "	46.57	58.32	79.8	......	21.5
75 (910)	" "	2 "	67.01	58.32	114.9	......	31.0
76	Sep. '87-Aug. '04	17 "	388.94	474.26	81.9	......	22.1
77	Sep. '99-Aug. '03	4 "	176.60	107.71	163.8	......	44.2
78	Sep. '00-Aug. '04	4 "	152.66	111.32	137.1	26.95	37.0

In comparing the isohyetose lines drawn by the writers with those given by the author (also shown on Plate LX), it will be noted that three general differences exist:

1.—The writers' 15-in. and 20-in. lines do not cross the valley summit between San José and Gilroy, a gap of more than 20 miles intervening between the two 15-in. lines in the valley.

2.—The writers show a mean seasonal rainfall of more than 50 in. at the crest of the Santa Cruz Range, as compared with the author's maximum of 45 in.

3.—The rainfall indicated by the local records falls off in quantity much more rapidly to the northeast of Mount Hamilton (Lick Observatory) than shown by the author's lines.

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TABLE 12.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

GROUP IV; GAUGES 79-97.

Using Mean of Gilroy, Lick Observatory, and Livermore Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with elevation).	Period of record.		Total Recorded Rainfall for Period of Gauge Record, in Inches.		Relation: Col. 3 / Col. 4	58-Season Mean, in Inches.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
79 (1400)	Sep. '03-Aug. '05	2 Seasons.	49.28	44.32	111.1%	21.88	24.3
80 (2200)	" -Aug. '04 / Sep. '05-Aug. '06	2 "	53.84	50.97	105.7	......	23.1
81 (2040)	" "	1 "	32.75	29.06	112.7	......	24.7
82 (2350)	Sep. '03-Aug. '04	1 "	24.80	21.91	113.2	......	24.8
83 (2350)	" -Aug. '05	2 "	58.08	44.32	131.1	......	28.7
84 (2460)	" "	2 "	58.93	44.32	133.0	......	29.1
85 (2350)	" -Aug. '07	4 "	133.24	105.20	126.7	......	27.7
86 (2450)	" "	4 "	139.59	105.20	132.7	......	29.0
87 (2900)	" -Aug. '05	2 "	57.08	44.32	128.8	......	28.2
88 (2950)	" "	2 "	53.81	......	121.5	......	26.6
89 (2250)	" "	2 "	52.63	......	118.8	......	26.0
90 (2275)	" "	2 "	56.81	......	128.3	......	28.1
91 (2250)	" "	2 "	56.35	......	127.2	......	27.8
92 (2350)	" "	2 "	52.74	......	119.1	......	26.1
93 (2300)	" "	2 "	54.88	44.32	123.9	......	27.1
94 (3625)	" -Aug. '04	1 "	29.00	21.91	132.4	......	29.0
95 (2330)	" "	1 "	24.57	21.91	112.1	......	24.5
96 (2650)	" "	1 "	26.75	21.91	122.1	......	26.7
97 (2550)	" -Aug. '05	2 "	39.54	44.32	89.2	21.88	19.5

The minor disagreements between many of the isohyetose lines of the writers and of the author are due, of course, to the greater number of data available to the former, and it is worthy of notice that the addition of more than 100 records within this area has made so few changes, and has so nearly confirmed the lines of the author. The general effect of the changes made is to show a greater mean seasonal rainfall for the Santa Clara Valley and the Santa Cruz Mountains to the westward, and a less rainfall for most of the Mount Hamilton Range to the eastward.

The correctness of the location of the 15-in. lines just east of San José, and in the extreme northeasterly portion of the area shown on Plate LX, is still comparatively uncertain from lack of adequate data fixing these lines. It is to be regretted that no rainfall records (so far as known to the writers, at least) have been kept in these localities.

Messrs. Haehl and Toll.

TABLE 13.—LONG-PERIOD MEAN SEASONAL RAINFALL DEDUCTIONS.

GROUP V; GAUGES 98–116.

Using Mean of Gilroy, San José, Livermore, Westley, and Lick Observatory Records as Basis.

(1)	(2)		(3)	(4)	(5)	(6)	(7)
Gauge No. (with elevation).	Period of record.		TOTAL RECORDED RAINFALL FOR PERIOD OF GAUGE RECORD.		Relation: Col. 3 / Col. 4	58-SEASON MEAN, IN INCHES.	
			At gauge.	Mean of base stations.		For base stations.	For gauge (deduced).
98 (2130)	Sept. '03–Aug. '05	2 Seasons	43.06	36.15	119.2%	17.94	21.4
99 (2150)	" "	2 "	42.05	.....	116.3	.....	20.9
100 (2550)	" "	2 "	38 67	.....	107.0	.....	19.2
101 (2160)	" "	2 "	38.29	.....	105.9	.....	19.0
102 (2110)	" "	2 "	40.79	.....	112.9	.....	20.3
103 (2250)	" "	2 "	33.45	.....	92.6	.....	16.6
104 (2450)	" "	2 "	36.51	.....	101.0	.....	18.1
105 (2290)	" "	2 "	35.05	.....	96.9	.....	17.4
106 (2140)	" "	2 "	38.46	.....	106.4	.....	19.1
107 (2250)	" "	2 "	43.18	.....	119.5	.....	21.4
108 (2120)	" "	2 "	36.33	.....	100.5	.....	18.0
109 (2030)	" "	2 "	42.91	.....	118.8	.....	21.3
110 (2450)	" "	2 "	54.21	.....	150.0	.....	26.9
111 (2100)	" "	2 "	42.06	.....	116.4	.....	20.9
112 (2500)	" "	2 "	40.70	36.15	112.6	.....	20.2
113 (1800)	" –Aug. '04	1 "	19.59	16.83	116.4	.....	20.9
114 (2000)	" –Aug. '05	2 "	41.08	36.15	113.7	.....	20.4
115 (2260)	" "	2 "	35.58	.....	98.4	.....	17.7
116 (2460)	" "	2 "	38.73	36.15	107.2	17.94	19.2

Mr. McAdie.

A. G. McADIE, Esq.* (by letter).—Amid much that is suggestive and of value in this paper and the discussion thereof, there are some statements which should be corrected or amplified, lest the engineering profession in general, and particularly the hydraulic engineers of the Pacific Coast, be led into serious error.

The author says: "So little rain falls from May 1st to the end of October that this period may be called rainless."

Based upon the records of the U. S. Weather Bureau, which have been maintained with fidelity, 10% of the seasonal rainfall in the district under discussion, for a period of 58 years, fell in the months which are called rainless by the author. In some seasons the rainfall in this rainless period has amounted to as much as 40% of the total. The year 1904 may be cited. This was an exceptional year, to be sure; but is not this the very information that is of most importance to engineers? Such a rainfall did happen, and may occur again.

* In charge of Local Office, U. S. Weather Bureau, San Francisco, Cal.

Mr. McAdie. Again the author says:

"Owing to the wide distribution of rain in ordinary rain storms, and to the freedom from local storms, the rainfall records at single stations are better indices of the amount of precipitation on large tracts than is ordinarily the case for records of rain in the East and in the Middle West."

As a matter of fact, owing to the topography of California, the precipitation is exceedingly variable, and if any generalization is to be made it would be that rainfall records at single stations could not safely be used as criteria for large tracts. The distribution of rain is more symmetrical and uniform in the East and Middle West than in the western portion of the country.

The author states that the rainfall is heavier on the southwestern slopes of the Coast Range than on the northeastern slopes. It is the commonly accepted view; but recent experiments lead us to doubt the truth of the statement. Recent rain measurements on a mountain near San Francisco showed the heaviest rainfall on the northeastern slope.* It is not on the western or southern slope, but just east or north of the crest line of the range, or summit of the peak, that condensation is greatest, for elevations below 5 000 ft. The rainfall will vary with the elevation and with the inclination of the rain-bearing winds.

The author says:

"Evidently, the moisture-laden air, as it rises up the mountain slopes, becoming cooler and losing density with increasing altitude, is forced to part with so much moisture before it reaches the mountain crest. * * * "

Cause and effect are mixed in this statement. When air cools it gains in density. The error in the statement is a natural one for an engineer to make. The widespread conception of rising air becoming cooler is erroneous. The opposite condition often happens. Indeed this inversion of temperature is the rule in summer afternoons in the region under discussion. It is true that the mixture of air and vapor ascending generally reaches cooler levels; but not always. The ascensional movement, however, is largely due to a considerable addition of heat. The density of air diminishes as the temperature rises, in the proportion of 1 to $1 + a\ T$, and also diminishes as the pressure decreases. Heat is utilized in doing the work of

* Comparative Rainfall at Mount Tamalpais, Cal., March, 1904. Observer, Mr. H. Legler, U. S. Weather Bureau; Station gauge located on top of ridge connecting east and middle peaks.

Gauge No. 1 on north slope of ridge, 30 ft. below top.

Gauge No. 2 on north slope of east peak, 250 ft. below the summit.

The experiments lasted one month, and measurements are given in detail on page 14, California Monthly Bulletin, March, 1904.

Station gauge caught 9.94 in.

Gauge No. 1 caught 26.50 in.

Gauge No. 2 caught 34.69 in.

Results: Gauge farthest northeast caught nearly four times the amount in gauge on south side of mountain. The winds were south and west.

expansion. This, however, is a question in physics. What the meteorologist particularly criticizes in this discussion of rainfall by the engineers is the entire absence of any reference to the amount of water vapor present and its behavior under change of temperature and pressure. This, which is an elemental factor in rainfall, is entirely ignored. There is no intimation throughout the entire discussion, many engineers participating, that the degree of saturation plays an important role in determining precipitation. A cubic meter of air is one thing and a cubic meter of saturated vapor and air is quite another. The addition of vapor increases the total pressure and causes an expansion of the volume when both are unconfined, as in ordinary free atmosphere. Mr. McAdie.

More serious, however, are the errors made both in the paper and in the discussion, in the construction of isohyets based upon fragmentary records. Discrediting the well-kept and extended official rainfall measurements made practically in one locality, and ignoring certain experiments made to determine variation in catch with change in elevation, these engineers expand their own unofficial records (which have never been subjected to the examination of Weather Bureau officials for reasons entirely unknown), and make of them long-range records of high accuracy presumably. Records covering but a few years are changed into records covering half a century or longer.

Nearly two pages are filled with criticism of the published Weather Bureau records. On pages 525 and 526 it is stated:

"No single record of San Francisco rainfall could be used for this comparison, the only available record extending back to 1849 (that published by the U. S. Weather Bureau) being a composite made up of records kept successfully at six different points in San Francisco."

As a matter of fact, the record from 1871 to 1908, known as the Signal Service-Weather Bureau record, was made practically in one portion of the city, and there have been but three removals of the gauge in a period of 38 years.

It is in the statements relative to a certain private rainfall record, however, that the greatest errors have been made. It is stated in the discussion:

"Careful rainfall records were kept in San Francisco by * * * under favorable and unchanging conditions."

And again: "Carefully kept by * * * at a single gauge, near the ground, under unchanging conditions."

It is known to the Weather Bureau, but evidently not to the engineers, that the location of the gauge in question has been changed many times with the owner's removal to different parts of the city. The points occupied differ by almost the width of the city. The particular record was kept by a gentleman who has for years compared his records with the Weather Bureau records. This seems to have been

Mr. McAdie. unknown to the engineers who made use of this record in the discussion of the paper, and they appear to assume throughout, and to build their whole argument upon, a superiority of the unofficial record. Much work done by them is thus rendered valueless.

These criticisms should put those who read the paper on guard against a too ready acceptance of the arguments given in connection with rainfall data.

One amusing illustration of blundering, in connection with rainfall in the section under discussion, occurred some years ago, and is referred to here only to put others upon their guard. The engineers of a certain water company obtained, from the Weather Bureau, records covering a certain water-shed. Later, the charge was made that serious errors existed in the published data. Investigating the matter, for it is realized that errors will creep into tables despite great care in their preparation, it was found that the engineers had obtained a copy of the original record at a particular point, and this, all uncorrected, they had reproduced. The retained copy in the Weather Bureau office showed that many corrections had been necessary. Indeed, there are very few rainfall records which do not need correction, especially if snow has to be entered. No acknowledgment of error, and no retraction of the charge, was ever made, so far as known. The uncorrected data may still be serving as a basis for the construction of isohyets and in the discussion of run-offs.

The measurement of rainfall is at best a difficult matter, simple though it seems. None realizes this better than the Weather Bureau official, and none is more ready to explain and allow for errors of exposure. There is no interest to be served but the truth. Yet it must be remembered that greater errors may sometimes result by hasty and ill-considered action in attempting to remedy existing imperfections than the errors due to these. For example, the decrease in the catch of rain on the roofs of high buildings due to eddy effects is an error of some importance; but the increase in catch of gauges on the ground, the interference with the gauge by thoughtless or designing persons, all too frequently reported, and the deflection of the air by surrounding objects unless the exposure is an ideal one (and this it is very hard to obtain in a city) cause errors much greater than those we are seeking to correct.

In recording and tabulating rainfall, let it be remembered that two separate and independent methods of checking are used by the Weather Bureau. Furthermore, if there is a marked departure from records at adjacent points, there is further inspection and examination. In all cases where published data come in conflict with unchecked private records the benefit of the doubt should go to the published data. It is a very wholesome experience for all hydraulic engineers personally to install a rain gauge, keep the records, and submit them for

examination to the nearest Section Director. One who does this will realize that in this seemingly simple operation one can make numerous and strange errors. Mr. McAdie.

C. E. GRUNSKY, M. AM. SOC. C. E. (by letter).—Aside from placing upon record the results of the rain and run-off studies near San Francisco, of some years ago, the purpose of the paper is two-fold: First, to emphasize the fact that these run-off studies, like many other run-off records when treated by averages, indicate a noteworthy regularity in the relation to the rainfall of the amount of water lost from water-sheds by evaporation, *i. e.,* evaporation from the surface of the earth, from trees and plants, together with water consumed in sustaining plant life; and second, to aid in establishing the laws of probable run-off for a large and important section of California. Mr. Grunsky.

Evaporation from an open-water surface in the heart of California's great central valley is, as stated, about 4 ft. per annum. At Lake Tahoe, at an altitude of 6 225 ft., it is probably between 30 and 36 in., and it will probably fall somewhere between 30 and 48 in., throughout the region under discussion. Although experiments made for the U. S. Department of Agriculture, by Professor S. Fortier, indicate that a saturated or moderately wet soil may give its moisture to the air more rapidly than an open-water surface, the total time each year during which soil is so wet that it will give off more water than a water surface, is small compared with the time during which the rate of loss is less than that from a water surface. Consequently, it seems safe to assume that the annual evaporation from water-sheds and the consumption of water by plant life nowhere in the United States will exceed the amount that would annually be taken up by the air from a water surface of equal area. There is thus set an upper limit for the loss of water into the air from the land and water surfaces of water-sheds, and it may safely be assumed that run-off will be at least equal to any excess of precipitation over water-surface evaporation. In such regions as California, however, where practically all rain falls between November 1st and May 1st, during which time the evaporation is only about one-fourth of the annual evaporation, the loss of water into the air will be very much less than the annual evaporation from a water surface. It may not be far wrong, as a first approximation, to assume it to be from one-half to three-fourths. It was this consideration, backed up by records of rainfall and stream flow, as these became available from time to time, that led to the formulation of the rule-of-thumb noted in the paper. The same consideration was used as a guide in shaping the run-off curves. The accordance of various long-time determinations of the amount of water lost from water-sheds by evaporation and related causes is so pronounced that it appeared tempting to try to establish some relation between this loss and the frequency and duration of rainstorms and the amount of

Mr. Grunsky. rain, but neither time nor the necessary detailed data for this purpose have been available.

The combination of rain-years of nearly the same amount of rain into groups, criticised by Mr. Duryea, was a convenience in constructing the curves. The result would not have been changed by using the annual records separately. Such use, however, would have shown the wide discrepancy of individual years. It can only be claimed for the curves, as stated in the paper, that they indicate probabilities. Mr. Duryea's statement that he has determined the relation of rain and run-off for a number of Sierra Nevada streams, long enough to give him 79 points—the rain-year being taken as the unit of time—and that all but 9 of these points indicate a greater run-off than the Sierra Nevada curve, must remain subject to further analysis. The rain distribution assumed for the several water-sheds from year to year may be subject to modification and likewise the interpretation of gaugings. It would be gratifying, however, to find that his opinion is correct, and that the curve shown for the Sierra Nevada streams is too conservative. The suggestion which he makes, that the error, resulting from the application of an average rainfall to entire water-sheds instead of to subdivisions thereof, is so small that it might well be disregarded, hardly needs comment. There would be no error if rain were uniformly distributed, neither would there be any error if the run-off curve were a straight line. As a result of the shape of the run-off curves, it follows that the error resulting from treating the water-shed as a whole is greatest when relatively large areas having small precipitation are included with areas in which the rainfall is moderately heavy. The examples selected by Mr. Duryea to show the insignificance of the error relate to water-sheds of small areas in regions of light rainfall; consequently, the error, although always present and always an error in the same direction, appears relatively small.

The non-agreement of recorded run-off with results obtained from the curve, in the case of Upper Crystal Springs Reservoir, which Mr. Murphy points out, should be considered in the light of the facts, that of the four water-sheds, especially considered, this is the one of least rainfall; that the total water yield is small; and that the error of 3 in. per year looks more formidable when expressed in percentage than it otherwise would. When practicable, all run-off curves should be checked by rainfall and stream measurements, and suitably modified for each water-shed.

The composite rainfall table for San Francisco, as prepared in 1903 and used in the study of run-off values, is not at hand at this writing and, therefore, cannot be added to the data already submitted. It was, as stated in the paper, a combination of five different records. These included, as a single record, the rainfall as measured by the U. S. Signal Service and by the U. S. Weather Bureau. The individual

records, such as those of Mr. Pettee and Mr. Tennent, necessarily differ materially in certain years from the composite. The latter, however, has the advantage of greater reliability than any of the individual records, whether in their original or corrected forms, as an index of rainfall in San Francisco, as well as of the general weather conditions in that vicinity. It is, therefore, more reliable as a basic table of rain than any table prepared by piecing out and correcting individual records. In substantially the form in which they were used in 1903, the rainfall records that were combined can be found in part in "Physical Data and Statistics for California."* Mr. Grunsky.

The rainfall data presented by Messrs. Haehl and Toll, in Table 7, are not convincing. For the years 1865 to 1892, they deduce a relation between the published record of the U. S. Weather Bureau and the Pettee record, and this relation is used in correcting the Weather Bureau records for the period from 1892 to 1907. The published record of the Weather Bureau, however, given in Column 2 of Table 7, is based upon rain measurements made in various localities, viz., near the foot of Market Street by Mr. Thomas Tennent until 1871; on the roof of the old Merchants Exchange Building by the Signal Service for several years; and on the roof of the old Phelan Building until 1892.

If the Tennent record be compared separately with the Pettee record, it will be found that (for 1865 to 1884, for which records are at hand) the Tennent record exceeds the Pettee record by 2.1 per cent. The Signal Service and Weather Bureau records, 1871 to 1892, exceed by 4.1%, the Pettee record for the same period.

These figures indicate that there is some uncertainty in regard to the amount by which rainfall in the heart of the business section of San Francisco exceeds the rainfall at points not specified, where Mr. Pettee's rain-gauge was located. Attention may also be called to the fact that for a part of the period, 1901 to 1907, the percentage correction claimed by Messrs. Haehl and Toll to be applicable to records established between 1892 and 1901 by rain-gauges on the eleven-story Mills Building, is assumed to apply also to the later records of the Weather Bureau, established by gauges on the new fourteen-story Merchants Exchange Building. The continuous record, worked out as in Column 6 of Table 7, like all corrected records, should be handled with due caution.

That Messrs. Haehl and Toll, in the construction of their isohyetals, covering a part of the area under discussion, have accepted the Morganhill and Coyote records in preference to the earlier longer record at Tennent is not surprising in the light of the additional data secured in that vicinity since 1903. The resulting changes in the position of the isohyetals affect but a small area to a moderate extent.

* "Physical Data and Statistics, California," William H. Hall, State Engineer, 1886.

Mr. Grunsky. Professor McAdie comments, from the standpoint of a meteorologist, on the method in which engineers have used the rainfall records at and in the vicinity of San Francisco. As he fears that some of the statements made may lead the engineering profession into serious error, a brief review of his criticisms will not be out of place.

Professor McAdie objects to the assertion made in the paper that the period from May 1st to the end of October "may be called rainless," and shows that about 10% of the annual rain falls in this time. It is cheerfully admitted that there was no need of specifying fixed dates between which, ordinarily, the rainfall is so light in the portions of California under consideration that it does not materially affect stream flow. On the other hand, the foundation for the statement will appear when it is recalled that the rain in the specified period at San Francisco amounts to less than 3 in., or, if stated by the month, to an average of about 0.4 in.; but, in order that the statements as made may be used with proper limitations, it is to be added that in some years the months of May and October, and perhaps once or twice in fifty years even September, are to be classed as wet months.

For San Francisco, based on the rainfall table published by Professor McAdie, in Bulletin "L" of the U. S. Weather Bureau, for the years 1849 to 1902, and extended to include 1907, the mean monthly fall of rain is 0.75 in. in May and 1.02 in. in October. There have been, in this 58-year period, 11 Mays with between 1 and 2 in. of rain, 5 Mays with between 2 and 3 in., and one May with between 3 and 4 in. One October is credited with 7.3 in., 3 with between 3 and 4 in., 6 with between 2 and 3 in., and 10 with between 1 and 2 in. In each of the remaining 78 Mays and Octobers, the rainfall was less than 1 in. The normal rainfall in the months June, July, August, and September is, respectively, if determined in the same way, 0.16, 0.02, 0.02, and 0.30 in.

Exception is also taken by Professor McAdie to the statement that "rainfall records at single stations are better indices of the amount of precipitation on large tracts" (in the part of California under discussion) "than is ordinarily the case for records of rain in the East and in the Middle West." In the use of the word "large" in this paragraph, vast areas were not in mind, but such areas as, in a State like California, may come under study as producers of run-off waters before reliable measurements of rain have been made at many points well distributed throughout the same. Professor McAdie says that if any generalization is made it would be that rainfall records at single stations could not safely be used as criteria for large tracts. This is true in a measure, depending on what is meant by "safely," but nothing is claimed for this method of estimating total quantity of falling water except approximation, the probability of ascertaining correct values decreasing, of course, with increasing areas. It is not

necessary to state that such generalization is hardly ever entirely satisfactory. Nevertheless, it will of necessity continue to be used. Mr. Grunsky.

Professor McAdie is no doubt correct in stating that the rainfall in the East and in the Middle West is more symmetrically and uniformly distributed than in the western portion of the country, if normals, determined by records extending over many years, are taken into account; and the statement in the paper, so far as it includes a comparison, should have been restricted to single storms and short time periods.

Notwithstanding Professor McAdie's statement relating to the result of an experiment, on Mt. Tamalpais, near San Francisco, the belief is adhered to that the heaviest rainfall, both in the case of the Coast Range and the Sierra Nevada, as indicated by annual normals, lies to the westward and not to the eastward of the crest lines of these ranges.

Professor McAdie deplores the absence of a discussion, in the paper, of the cause of rain. This would have been entirely foreign to its purpose, and therefore could find no place in it. Whether the statement made relating to the changes resulting from increase of altitude follows the rule suggested in the paper or whether inversion of temperature is the commoner phenomenon may be submitted without further discussion.

He condemns the practice of estimating by expansion the probable normal rain at a station from records that do not cover the entire period taken into account, that is, by applying a correction ascertained from some complete nearby record. The result of doing this, as shown in the paper, speaks for itself.

His statement that the location of the Pettee gauge, referred to by Messrs. Haehl and Toll as having been kept "near the ground under favorable and unchanging conditions," has been changed many times with the owner's removal to different parts of the city, is specially noteworthy and of importance to those who have occasion to use this rain record.

There is nothing in the paper reflecting upon the reliability of data furnished by the U. S. Weather Bureau. The work done by that Bureau is valued by the entire profession, and the results of its observations are accepted without question. Nothing that has been said should be considered as an unfavorable or unfriendly criticism of the Bureau or its work, or as detracting in the slightest degree from the importance thereof. It must, however, be left to the engineer who has occasion to use the recorded facts relating to rainfall, whether observed by the Bureau or by private parties, to take into account all circumstances attending their observation.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1091

ADDRESS AT THE 40TH ANNUAL CONVENTION, DENVER, COLORADO, JUNE 23D, 1908.

BY CHARLES MACDONALD, PRESIDENT, AM. SOC. C. E.

DENVER, 1886-1908.

Twenty-two years ago the American Society of Civil Engineers held a Convention in this City upon the joint invitation of their Denver brethren, the Denver Chamber of Commerce, and His Excellency, Governor Evans, of Colorado.

The recollection of that memorable occasion has remained so fresh as to have inspired the desire to repeat the visit in order that we may more fully appreciate the tremendous strides that have been made in this brief interval of twenty-two years. Denver was young in 1886—with scarcely 65 000 people all told, but with evidences of power and potency which gave promise of greater things to come, and these are the things which we are now permitted to examine for ourselves, and to offer our congratulations on the wonderful results accomplished.

To mention but a few of them: the population has increased threefold in twenty-two years; the assessed value of property, which was \$30 610 330 in 1886, is \$122 585 925 to-day; and the best evidence of prosperity is to be noted in the increase in bank deposits from \$9 000 000 to \$40 000 000. It would be foreign to our purpose to enlarge upon the records of progress everywhere in evidence; suffice it to say, that

the controlling reason which induces the Society to meet in Convention annually in different parts of the country is to present object lessons such as this to its membership in order that as an organization it may be kept fully abreast of the times.

American Society of Civil Engineers, 1886-1908.

An examination of our own rate of progress during the past twenty-two years furnishes gratifying evidence of the success of the policy pursued by the management of the Society.

In 1886, the total membership was 925, of which there were but 6 in the entire State of Colorado, while at present the membership is upwards of 4 600, of whom 40 are residents of this State, 19 being located in the City of Denver.

The Society first began the publication of papers in 1867, but it was not until November, 1873, that regular monthly publications were issued. The current volume of *Transactions* completes the 66th volume of a library which is second to none in professional interest, a result which is largely due to the untiring energy and administrative ability of our Secretary, Mr. Chas. Warren Hunt.

Function of the Engineer.

The question naturally arises, what have engineers done since 1886 which is of special signification? Perhaps the most important result of engineering activity is the increase in railway mileage from 137 615 to 235 409 in the United States alone. When it is considered that the total mileage of railroads in the world at the present time is approximately 570 000, the proportion constructed by American engineers is, to say the least, creditable. The result of this enormous increase in facilities for internal communication has developed problems in Interstate Commerce which have put a strain upon the Constitution of the United States itself, as well as upon those of the several States, which bids fair to approach almost up to the elastic limit.

To construct a railroad involves a knowledge of Nature's laws. It is an exact science, and the result should insure the development of traffic upon lines of least resistance, and, consequently, at the least cost; but judging from the experiences through which we are passing, the problems of utilizing these mighty engines in developing the resources of the country through which they pass, have not been treated

with that due regard for the immutability of natural law that has characterized the work of the Engineer.

Conflicts have arisen between the sovereigns by whom powers were granted to construct the properties, and the managers of these properties, as to the right to fix rates of transportation upon them, resulting in a serious paralysis of business throughout the country, which, for the time being, threatens to retard further development until these differences are settled upon some equitable basis.

A careful examination of the methods employed in fixing traffic rates must convince liberal observers that no general principle of law has obtained. Traffic managers have too often been governed by their individual opinions as to what the rate should be, and the amount of business they could secure, irrespective of the interests of the people as a whole from whom they originally derived the right to fix any rate. The result has been, as might have been expected, discrimination favoring certain localities and individual shippers, which has aroused the whole country to the necessity of framing such laws as will protect the rights of all the people in the use of the franchises which have been granted by their representatives.

It has been said that the construction of a railway is an exact science, and while it cannot be claimed with equal propriety that the management of it can be formulated on equally exact lines, it is nevertheless the duty of those in control to investigate the general principles involved, which, if applied, will result in establishing harmony where now deplorable conflict exists.

In this connection, what is the function of the Engineer? Heretofore, with few exceptions, he has been content to fulfill the duty of executive officer merely, in obedience to instructions from the financial interests controlling, without regard to questions of political economy involved in subsequent management.

This is not the highest ideal to which he should attain. To be an engineer of material only is to be but a subordinate element in a profession which has contributed, and will continue to contribute, more to the advancement of civilization than all others combined. Rather should he strive to become an engineer of men, pointing out lines of activity, based upon scientific principles, which permit of no discriminations or unfair advantages to favored interests. There should be no guesswork in his study of problems of transportation, but he should have

ever before him the example of such men as the late Albert Fink, Past-President, Am. Soc. C. E., who as early as 1874 was able to evolve a system for the management of traffic in this country, which if it could have been crystallized into the law of the land, would have been hailed by all parties in interest as an equitable deliverance from such disastrous complications as now beset us.

Again, in the management of employees, the Engineer is best equipped by education and experience for settling differences between employer and employed. A single instance will suffice to illustrate. A recent predecessor of mine, then an officer of one of the most important trunk lines in the country, was called upon by the representatives of the Boilermakers' Union on the line of his road, who, with imperious swagger, demanded an immediate advance in wages. There was to be no discussion of the subject, and unless their demands were acceded to a strike would follow. It is needless to refer to what actually did follow, but the result was that every one of those men who came with blood in his eye, departed with the humble request that they might be permitted to shake the hand of a gentleman.

We must evolve more men of the type above referred to if we would render the best account of our stewardship.

Conservation of Natural Resources.

The late Lord Kelvin, when in this country some years ago for the purpose of advising as to the best means of utilizing the water power at Niagara, was asked what effect the withdrawal of so much water from the Falls would have upon the natural beauty of the cataract. His reply was that of the true engineer: "What has that got to do with it? I consider it almost an international crime that so much energy has been allowed to go to waste."

A similar question arose in connection with the construction of the dam across the Nile at Assouan. Archæologists all over the world raised a protest against the project, because, forsooth, the ruins of the Temple of Philæ would be submerged for six months in the year. I venture to say that of all the visitors to Egypt not one in a thousand but would far rather inspect the graceful lines of that superb work, and picture to themselves the inestimable benefits accruing to the hundreds of thousand of tillers of the soil in the valley below, than gaze for a few passing moments upon the crumbling remains of

a Grecian temple two thousand years old. Engineers will naturally consign all such archaic questions to the oblivion of the past, and concern themselves with that which confers the greatest good upon the greatest number. This line of thought naturally leads to the consideration of a subject which is attracting special attention at this time.

The President of the United States, in a letter dated January 14th, 1908, invited John Hays Hammond, President, American Institute of Mining Engineers; Minard Lafever Holman, President, American Society of Mechanical Engineers; Henry Gordon Stott, President, American Institute of Electrical Engineers, and the speaker, to attend a Conference of the Governors of the several States, with their experts, to be held at the White House, May 13th-15th, for the purpose of considering the question of the Conservation of the Natural Resources of the Country.

At this conference these four Presidents of Engineering Societies, representing approximately 20 000 American engineers, recommended the following resolutions for adoption by the Conference:

"*Resolved:* 1. That this Conference places on record its conviction that to conserve and protect from waste and destruction the natural wealth of the United States in mines, forests, lands, and waters is of vital necessity to the public welfare. Action in this matter has been too long delayed, and vast loss has resulted in consequence, notably in the destruction wrought by forest fires, by floods, and the ruin of lands whose fertility and crop-bearing power has been lost. This unfortunate destruction of part of the natural wealth with which this virgin continent was originally stored makes it all the more necessary that wise action be taken to check further loss.

"2. Though it recognizes the imperative need for prompt action, this Conference is impressed with the difficulty of framing legislative acts which shall result in the largest measure of public benefit. The problems presented are many of them new and unprecedented. It is probable that action by both the Federal Government and the individual States will be essential, and it may also be possible by suitable laws to enlist the aid of private enterprise. But to decide upon the proper distribution of responsibility and to frame laws which shall not work injury as well as benefit is a matter demanding most careful study and investigation by men of high standing and expert qualifications.

"3. While certain individual measures may be already in such shape that action upon them may wisely be taken, this Conference holds that for the guidance of legislators, both State and Federal, a thorough investigation and study should be made by National and State Com-

missions so constituted that their conclusions and recommendations will be everywhere recognized as authoritative and made solely in the public interest.

"4. This Conference, therefore, urges upon Congress and the State Legislatures the enactment of laws authorizing the President and the Governors, respectively, to create National and State Commissions to investigate and report upon what measures should be taken to conserve the National and State natural resources.

"These commissions should report at the earliest possible date consistent with the thorough performance of their work, in order to enable the President and the Governors to transmit with recommendations their reports to Congress and the State Legislatures for such action as may seem advisable to protect our natural resources from further spoliation and destruction, and to secure such economy in their use as will preserve for coming generations the foundations of prosperity.

"5. In order to insure the harmonious co-operation of all the Commissions, this Conference requests the President to call another National Conference at such time as may seem most advisable.

"6. To secure the most efficient organization for handling the National problems which the reports of these Commissions will inevitably raise, this Conference recommends for the consideration of the President and Congress the formation of a Department of Public Works to which these and other engineering matters could be referred, and to which the State Commissions could apply for information and assistance."

These resolutions were duly referred to the Committee on Resolutions of the Conference, and were favorably received by that body.

It will be observed that the crux of the recommendations rests in Clause 6—advising the formation of a Department of Public Works, which would naturally include under its jurisdiction all works of Public Utility classed under the heads of Improvement of Waterways and Harbors, Irrigation, Hydraulic Development, Drainage, etc., now distributed through the Departments of War, Interior, Agriculture, and Commerce and Labor.

At the time it was written, we were not aware that a bill had already been introduced by the Hon. J. E. Ransdell, of Louisiana, having this very object in view, and that in his introductory speech very powerful arguments had been given in favor of its passage. It was scarcely to be expected that this bill could be passed at this session, but members are earnestly requested to read carefully the arguments of the Honorable Gentleman from Louisiana, and if consistent with their own views, to urge upon their Representatives the passage of such a bill,

The general result of the Conference has been to arouse public interest as it never has been roused before, and it is to be hoped that the engineers of the country will realize their opportunity to exert a potent influence in framing such legislation as may best be suited to the accomplishment of the desired ends.

Agriculture.

Having become a farmer myself, it may perhaps be permissible to refer to the possibilities of adding a new qualifying adjective to the subdivisions of the profession. If it can be proved that two blades of grass can be grown where one has heretofore been found to be the limit, it is certain that the sources of power in Nature have been scientifically utilized, and the general wealth of the country correspondingly increased. This of a surety can be accomplished if the engineer will but turn his attention to Agriculture as an adjunct to the creative work with which he has become familiar in the construction of lines of transportation, irrigation and drainage.

Quoting from the admirable paper read before the recent Conference at Washington by James J. Hill, F. Am. Soc. C. E.:

"In no other important country in the world, with the exception of Russia, is the industry that must be the foundation of every state at so low an ebb as in our own * * *. Our soil, once the envy of every other country, gave an average yield for the whole United States, during the ten years beginning with 1896, of 13.5 bushels of wheat per acre. Austria and Hungary each produced over 17 bushels per acre, France 19.8, Germany 27.6, and the United Kingdom 32.2 bushels per acre."

This is an appalling statement which, unfortunately, cannot be controverted, but the explanation is simple, and the remedy easily obtainable. We have been single cropping our soils until they have become unsanitary, and we have failed to make use of suitable methods of cultivation. In corroboration of this, I quote again from Mr. Hill's paper:

"At the experiment station of the Agricultural College of the University of Minnesota, they have maintained 44 plots of ground, adjoining one another, and as nearly identical in soil, cultivation and care as scientific handling can make them. On these have been tried, and compared, different methods of crop rotation and fertilization together with single cropping. The results of 10 years' experiments are now available. On a tract of good ground sown continuously for 10 years to

wheat, the average yield per acre for the first five years was 20.2 bushels, and for the next five 16.92 bushels. Where corn was grown continuously on one plot, the average yield was 16 bushels, while on an adjoining plot where corn was planted but once in five years in a system of rotation, the average yield of the latter for the two years it was under corn was 48.2 bushels per acre."

Evidence such as this should not be ignored, especially by the engineer who would aspire to control the properties which he has created. It is not enough that he should construct new lines of communication whereby virgin soil can be rendered productive, but he must furnish the incentive to would-be settlers to maintain and improve the productive capacity of the soil rather than allow it to become exhausted by improper cultivation.

This is a simple commercial proposition. If the products of the soil are allowed to decline steadily, the freight handled over tributary lines of communication will as certainly follow suit, and inducements to capital, for further extensions, will prove more and more difficult to obtain.

Engineering Ideals.

In whatever branch of the profession in which we may be engaged, whether we call ourselves Civil, Mechanical, Mining, Electrical, Military, Naval, or even Agricultural Engineers, the only safe guide is a strict adherence to Nature's law of least resistance. From the beginning of time, every particle of matter has followed the exact resultant of contending forces, without deviating a hair's breadth from a course which has permitted it to attain a position of equilibrium with the least expenditure of energy, resulting in the most expressive, and therefore impressive, effects.

Whether it be in the growth of a tree, the flow of a river, or the slow but certain modifications of mountain forms, the operation of this law of Nature has resulted in the combination of the beautiful with that which is good and true. If, therefore, the Engineer would attain to the highest excellence in his profession, he must so utilize the sources of power in Nature as to effect the greatest good at the least cost.

He must combine strength and durability with attractive outline and artistic excellence. It is not sufficient that a bridge, for example, shall be strong enough to carry its load. Any engineer who is familiar with the laws of statics can calculate the strains, and proportion the

material for a given diagram of a bridge in which the number of panels and relation of height to span is taken arbitrarily. The structure so proportioned will be as strong as may be desired for the purpose, but if the diagram so chosen does not present a pleasing appearance, it will be found that more material has been expended than would have been the case if a more artistic arrangement had been adopted. Again, if in cases where the number of piers is not restricted, the proper relation of cost of superstructure to cost of substructure has not been observed, the total cost of the completed structure will be in excess of scientific requirements.

It is true that what is called Art has not been considered a fixed science, which is equivalent to saying that the rules which govern artistic construction are not capable of mathematical demonstration. Nevertheless, I believe it will be found to be the fact that the structure which has been designed upon the most scientifically accurate proportions, that is to say, which accomplishes the object for which it was intended in accordance with Nature's great law, will present the most pleasing outline, and that eventually the conception of the Engineer and the Architect will merge into that of the true Artist and Engineer, of whom the immortal Michael Angelo was the great prototype.

MEMOIRS OF DECEASED MEMBERS.

CHARLES HAYNES HASWELL, Hon. M. Am. Soc. C. E.*

DIED MAY 12TH, 1907.

A life of remarkable professional activity, characterized by conspicuous public service, came to its close when, on May 12th, 1907, Charles Haynes Haswell died at his home, 324 West 78th Street, in New York City.

Mr. Haswell was undoubtedly the oldest civil engineer in the world, and had he lived ten days longer, he would have entered upon his 99th year. He was elected a Member of the American Society of Civil Engineers on January 29th, 1868, and on May 12th, 1905, he was made an Honorary Member, being one of the forty men upon whom this distinction has been conferred since the organization of the Society.

A son of Charles Haswell, a native of Dublin, who was in the British diplomatic service, and of Dorthea Haynes, a member of a prominent family in the Barbadoes, he was born on May 22d, 1809, in a house which is still standing on North Moore Street, in New York City. His education was obtained in the best New York schools of the time, and was liberal or classical in its character, as no school of applied science had yet been established in the United States.

At the age of nineteen he entered the service of James P. Allaire, who was the owner of what was then the greatest steam engine works in the United States. By close application, he acquired a practical and thorough knowledge of mechanical and marine engineering, and his excellent work coming to the attention of the United States Navy Department, he was, in 1835, employed to prepare designs for the machinery of the United States Steam Frigate *Fulton*. He had the satisfaction of superintending the construction of the engines and boilers of this vessel, as Chief Engineer, under a commission signed by President Jackson. Afterward, he designed or superintended the building of the war-ships *Missouri, Mississippi, Michigan* and *Allegheny,* and a number of revenue cutters.

In 1843 Mr. Haswell was appointed the first Engineer-in-Chief of the United States Navy, his administration of which office was characterized by a devotion to the highest professional ideals, absolute integrity, and rare efficiency. His uncompromising fidelity to duty, as interpreted by his ideals and standards, was not always appreciated by those with whom he had official relations, and in 1851 he resigned from the naval service to engage in private practice in New York City, as a civil and marine engineer. The modern steam yacht is said to have been created by Mr. Haswell, as the *Sweetheart,* which was probably

*Memoir prepared by Nelson P. Lewis, M. Am. Soc. C. E.

the first vessel of this type, was designed and built by him in Brooklyn, some time before 1840.

While best known through his connection with steam and marine engineering, his work covered nearly all branches of civil and mechanical engineering, and as the author of the "Engineers and Mechanics Pocket-Book," which bears his name, he was known throughout the world. This book was a standard work of its kind for more than sixty years, and passed through seventy or more editions. He was working at his desk upon material for a new edition, when, rising from his chair, he fell and sustained injuries which resulted in his death the following day.

During the Civil War, Mr. Haswell was an enthusiastic supporter of the Union cause. Not only did he go to the front as the representative of a "Committee of Citizens of New York" and direct the disbursement of funds raised by the Committee, a mission requiring much tact and discretion, but he was in active service under General Burnside, who recognized his excellent work in his reports to the Secretary of War.

His interest in the affairs of the city which was his birthplace and home was always keen and unselfish. From 1855 to 1858 he was a member of the City Board of Councilmen, and, during his last year of service, he presided over that body. He served as member of a number of important commissions, and was one of the Trustees of the Brooklyn Bridge. From 1898 until the time of his death he was Consulting Engineer to the Board of Public Improvements and the Board of Estimate and Apportionment. He personally made the plans for and supervised the installation of the heating and power plants for the public institutions on Hart's Island, and prepared plans for the enlargement and improvement of Riker's Island. Until within a few months of his death, he was regular in his attendance at his office, where a large part of his time was spent over the mahogany drawing board, concerning which Mr. Haswell wrote in 1904:

"It has been in use 53 years without requiring to be trued. On it was executed the feat that has become historical both here and in Europe, that of the delineation of the entire working drawings of the engines and boilers of the U. S. Steamer *Powhatan,* cylinders 70 inches by 10 feet stroke of piston, the demands upon my time not admitting of the delay of making a general drawing before furnishing those of the detailed parts."

In the summer of 1904, then in his 96th year, he was retained as an expert to examine and report upon a boiler plant in Chicago, where he spent a week in making tests and preparing his report.

Appreciation of Mr. Haswell's professional work was not confined to his own country. More than half a century ago the Czar of Russia sent him a diamond ring, accompanied by an expression of his thanks for services rendered to the Imperial Government in sending to it a

number of plans and drawings. The engineers and naval architects of Great Britain have frequently indicated their high regard for and deep obligations to him, and during the visit of members of the Institution of Civil Engineers to America in 1904, he was the recipient of conspicuous attention from them and their President, Sir William H. White. Those who went to West Point with the visiting engineers will recall the fact that he walked unaided, down the long line of cadets, with the reviewing officers and guests, and his tall, erect figure was the most prominent in the party.

He contributed several papers and discussions to the *Transactions* of this Society and to the *Minutes of Proceedings* of the Institution of Civil Engineers. In 1897 he published his "Reminiscences of an Octogenarian," a book which, while lacking continuity of narrative, gives some admirable and interesting sketches of New York City between the years 1816 and 1860, and affords evidence of the refined tastes and admirable public spirit of the author.

Owing to Mr. Haswell's great modesty, he rarely spoke of his personal achievements or of incidents in which he figured, and it was difficult to realize the important services which he had rendered to his profession and to his country. On the rare occasions when he would indulge in conversational reminiscences, he was delightful. His early education, as already noted, was of the liberal sort, and there was a refinement in his manner and conversation which showed the influence of that training. He was familiar with the best literature, and his Latin quotations, while used without a suggestion of pedantry, frequently gave force to his illustrations and charm to his conversation. His bearing toward his associates and toward those who were many years his junior was characterized by a gentleness and uniform courtesy which made him a delightful companion and an always welcome visitor, while his tall, slender figure made him conspicuous in any assembly.

With his keen appreciation of the dignity of his profession, his high sense of personal honor, and his rare consideration for the feelings of others, he was an admirable example of the old-school, courtly gentleman of the type which has become all too rare.

Mr. Haswell, in addition to his membership in this Society, was also a member of the following technical and social organizations: Honorary Member of the American Society of Mechanical Engineers; Honorary and Life Member of the Institution of Civil Engineers; and a member of the Institute of Naval Architects of Great Britain; the American Society of Naval Engineers; the Municipal Engineers of the City of New York; the American Institute of Architects; the New York Academy of Sciences; the New York Microscopical Society; the Society of Authors; the Engineers' Club of New York; the Engineers' Club of Philadelphia; and the Union Club of New York.

GEORGE IRVING BAILEY, M. Am. Soc. C. E.*

Died March 28th, 1908.

George Irving Bailey was of English and Holland Dutch descent. Dr. Solomon Bailey, his grandfather, was for many years a prominent physician in Bethlehem, Albany County, New York. Dr. James S. Bailey, of Albany, his father, was one of nine children, and was graduated at the Albany Medical College in 1853; he practised in Alabama and Texas, and was a surgeon in the Confederate Army between 1861 and 1866, after which time he practised in Albany, New York, until his death, July 1st, 1883. Dr. James S. Bailey was married to Frances J. Keith, of Augusta, Georgia. The subject of this memoir was a great-great-grandson of John Keith, a colonial soldier who fought in the Battle of Lexington.

George I. Bailey was born at Hempstead, Texas, on December 24th, 1861, and was graduated at the Albany High School in the class of 1880.

He entered the Department of the State Engineer and Surveyor in September, 1880, was a rodman up to November, 1884, then leveler on construction until November, 1885, then draftsman in the Department, and was appointed Assistant Engineer in Charge in September, 1887, and so continued until June, 1892. As Assistant Engineer, he had charge of the lengthening of a number of the locks in the Erie Canal, designing and making plans for and building the culverts, waste-weirs, retaining walls, and iron bridges along the line of the State's canals, and making miscellaneous surveys for the State, and defending suits against it.

He was in charge of the survey for the storage dam across the Genesee River, and conducted tests for the tensile and crushing strength of concrete in connection with this dam. These tests were among the earliest of this character. He was appointed Superintendent of Water-Works of Albany, New York, in June, 1892. On May 16th, 1892, a new Board of Water Commissioners had been appointed, and the late Elnathan Sweet, M. Am. Soc. C. E., former State Engineer and Surveyor, having been chosen as its President, requested Mr. Bailey to sever his connection with the State work and accept this office. Mr. Bailey held the position of Superintendent of Water-Works in Albany until February, 1902, and while in that office some of the most valuable work of his life was accomplished, the complete record of which is fortunately preserved in his annual reports, which are models of painstaking application.

He was at first engaged with his routine duties, but, at the same time, was deeply interested in the proposed new water supply from

* Memoir prepared by Horace Andrews, M. Am. Soc. C. E.

Kinderhook Creek, and attended to much of the work necessary in developing the scheme. This plan, however, was denied support, the necessary legislative action being withheld, and, acting under a new Board of Water Commissioners which supplanted that of which Mr. Sweet was President, he assumed charge of the installation of new pumps for increasing the supply of raw water from the river by 15 000 000 gal. per day. In connection with this work, he planned and supervised the laying of 11 200 ft. of 30-in. main. He continued his interest in the Kinderhook Creek development, which he hoped might supplant the use of the river water, and supervised the continued weir measurements for that source of supply, reporting to the new Commission, on January 1st, 1895, in regard to these weir measurements, as follows:

"An examination of these records will show that, notwithstanding the protracted and severe droughts of the summer, all claims made as to the sufficiency of the creek as a means of supply have been sustained."

In Mr. Bailey's fifth report, dated January 30th, 1897, the matter of the filtration of the river water was first discussed by him. The city was at this time hopelessly involved in the continued use of the water of the Hudson River as its chief source of supply, and it was apparent that filtration was the only resource for improving the unsanitary condition of the water. Mr. Bailey made personal visits to various filtration plants, and the matter was finally submitted to Allen Hazen, M. Am. Soc. C. E., in consequence of whose recommendations the slow sand filters were adopted in preference to mechanical filters with chemical precipitation of solids, which had strong advocates. The slow sand filter project was recommended to the Common Council on February 13th, 1897, the estimated cost being \$478 000. The plans were approved on April 19th, of the same year, and Mr. Hazen was appointed Chief Engineer, and, under the joint supervision of Mr. Hazen, Mr. Bailey, and the Water Commissioners, the filters were successfully completed.

Mr. Bailey's seventh, eighth, ninth, and tenth reports are filled with valuable descriptions of the construction of the filters, and with accounts of their operation and of the consequent decrease in the typhoid death-rate in the City of Albany. These reports, relating to the development of the filtration works—which were regarded as models, and visited from all parts of the United States—are of the highest value to the engineering profession. They are profusely illustrated, and, in their entire preparation, show Mr. Bailey's interest and unremitting care.

Upon the cessation of Mr. Bailey's superintendency of the Albany Water-Works, he undertook work for the development of an ice plant near Albany, utilizing the overflow of the Maezlandt Kill, one of the

oldest sources of Albany's water supply. The ice-collecting ponds planned by Mr. Bailey have been in successful use since their completion by him in the summer of 1902. The upper pond has a concrete dam 30 ft. in height, while the lower pond is formed by the construction of an extensive dam of earth with a maximum height of about 18 ft.

Late in the fall of 1902, Mr. Bailey was called to New York City to undertake the completion of some defaulted contracts in the Borough of the Bronx. These were administered so satisfactorily to his employers that his relations with them continued to be of a cordial nature until his death. Mr. Bailey resolved to continue the business of a contracting and consulting engineer in the Borough of the Bronx and did so with great success until the close of his career. He secured contracts and successfully completed in that Borough the regulating, grading, curbing and flagging of many streets, avenues and roads. He also constructed sewers in Boston Road and in Bryant Avenue. At the time of his death, he had under way, and about one-third completed, a water filtration plant in Yonkers, New York.

In 1898, Mr. Bailey married Mrs. Eva Fish (née Williams), of Utica, New York. Four children were born to them: William Weaver, Lois, Helen, and Catharine.

While apparently in perfect health, he was stricken with heart trouble at his home in the Bronx on the afternoon of Friday, March 27th, 1908, and died the next morning before his brother, Dr. Theodore P. Bailey, could reach him from Albany. He is survived by his widow and his four children, by his mother, one brother, Dr. Theodore P. Bailey; two sisters, Mrs. John R. Kaley and Miss Corrine Bailey, all of Albany, New York.

Mr. Bailey was elected a Member of the American Society of Civil Engineers on October 1st, 1890. He joined the New England Water Works Association on December 14th, 1892, and continued his connection with that Association until December 31st, 1906. He was a member of the Masonic fraternity, having attained the distinction of the thirty-second degree. He was elected a member of the Albany Club on January 28th, 1893, acted as its Secretary from May 1st, 1901, to May 1st, 1903, and resigned on October 1st, 1903. During the past few years he was a member of the Contractors' Association, in the Bronx, and was active in its development. He was also a member of various social organizations in New York City.

Mr. Bailey's contributions to engineering literature are entirely in connection with his water-supply work. In addition to the valuable reports comprised in the volumes of municipal records of the City of Albany, he wrote several communications for *Engineering News*. His discussion* of Mr. Hazen's paper, "The Albany Water Filtration

* *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 296; an abstract of this discussion may be found in the *Journal* of the New England Water Works Association, Vol. 14, p. 329.

Plant," relates to the operation of the filters. Mr. Bailey contributed to the *Journal* of the New England Water Works Association an article concerning the care of fire hydrants in winter. His article entitled: "The Effect of Water Meters on Water Consumption in the Larger Cities of the United States," was reprinted in the *Journal* of the New England Water Works Association in June, 1901, the editor stating that the paper contained so much valuable information as to warrant its republication without apology. At the 1901 Convention of the American Water Works Association Mr. Bailey presented a paper on "The Albany Filtration Plant, and Some of the Results Obtained."

Mr. Bailey was a diligent and persevering worker, and obtained the success that comes from painstaking application. His friends testify to his personal worth: C. A. Crane, Assoc. M. Am. Soc. C. E., writes: "Mr. Bailey was universally esteemed by all the contractors and city officials with whom he came in contact to an unusual degree," and this writer closes by saying: "thanking you for the opportunity to do what little I may in memory of one for whom I felt the greatest affection."

Chapman L. Johnson, M. Am. Soc. C. E., a friend of long standing, says:

"He was one of my best and dearest friends, and the news of his death is a sad and severe shock to me. Mr. Bailey was a man of indomitable energy and will power and of the strictest integrity. His success was due to persistent insistence on theories arrived at and from inflexible integrity of motive."

Mr. Hazen, who was so long his associate, adds: "Mr. Bailey was a big-hearted man, a good friend, and one who could always be counted on."

JAMES DUN, M. Am. Soc. C. E.*

DIED FEBRUARY 23D, 1908.

James Dun was born on September 8th, 1844, in Chillicothe, Ohio, and there his early education was obtained. After being graduated from the Chillicothe Central High School, he attended a private school at St. Catherines, Ontario, Canada. Later, he returned to Ohio and received his finishing education at Miami University, Oxford, Ohio.

Mr. Dun began his professional career in 1866, as a member of an engineering corps working near Indianapolis, Indiana; later, he was Instrumentman on the survey for the old Louisiana and Missouri River Railway, between Louisiana and Cedar City, Missouri, now a part of the Chicago and Alton System.

In 1867 he was appointed Assistant Engineer of the Atlantic and Pacific Railway, under Mr. Thomas McKissock, Chief Engineer. This road is now a part of the Frisco System.

From 1871 to 1873 Mr. Dun was Assistant Engineer of the Missouri Pacific Railway, under Mr. James W. Way, Chief Engineer. From 1874 to 1877 he was Chief Engineer of the Union Depot Company, and built its yards, and freight and passenger station in the vicinity of Twelfth Street, St. Louis, Missouri. In 1877 he was appointed Superintendent of Bridges and Buildings of the St. Louis and San Francisco Railway Company, and in 1878 was appointed Chief Engineer of the same system, also filling, for a part of the time, the position of Acting General Manager, during the last illness of Mr. C. W. Rogers, Vice-President and General Manager.

In 1890 Mr. Dun was appointed Chief Engineer of the Atchison, Topeka and Santa Fé Railway, and in 1900 was appointed Chief Engineer of the Santa Fé System. In 1906 he was appointed Consulting Engineer of the same system, which position he held at the time of his death, which occurred on February 23d, 1908, at St. Augustine, Florida.

Mr. Dun was elected a Member of this Society on June 7th, 1876. He was also a member of various other technical societies, including the Engineers Club of St. Louis, and the Western Society of Civil Engineers, of Chicago.

His professional reputation was international, and the Frisco and Santa Fe Systems show to-day the characteristics of his work. The writer sustained social and official relations with Mr. Dun from 1869—not continuously, but nearly so—and in all those years has never seen Mr. Dun's enemy nor heard an unkind criticism.

Broad-gauged, liberal-minded, and with spotless integrity, he recognized his work and performed it to the letter. Accurate and resource-

* Memoir prepared by J. F. Hinckley, M. Am. Soc. C. E.

ful, with a keen mind for details, and a fund of information acquired by long and varied experience, he was especially well qualified to sustain the confidential relations with his superior officers which he held for so many years before his death. Loyal, kind and generous, he was a most charming companion.

Born a gentleman, he developed into the highest type of manhood. Full of sympathy for all with whom he came in contact, he was never too busy to give time to the trouble of his friends, nor counsel to the young graduate who was looking for an opportunity.

One of his friends—a prominent railroad man who had known him intimately for more than thirty years—writes as follows:

> "I never knew him to do a wrong. His integrity was like the sun's rays; it came swift from a soul of fire. Nothing deflected him from the straight course. * * * He sympathized with all men in trouble, and appreciated the infirmities of human nature; but he never could understand why men were dishonest. The men in this world who have never been swerved from the right by either passion or covetousness have been few: James Dun is the only one I ever knew."

The high esteem in which he was held by his associates cannot better be summed up than in the words of an old roadmaster who worked under his direction for many years; upon being told of Mr. Dun's death, he remarked: "There may have been a better man, but I never met him."

JOHN EDWIN EARLEY, M. Am. Soc. C. E.*

Died December 20th, 1907.

John Edwin Earley was born at Walpole, Massachusetts, on December 31st, 1842, and was of Puritan stock, his father being John Earley, and his mother Sarah Otis, of Scituate, Massachusetts. He was fitted for college at Phillips (Exeter) Academy, and was graduated from Union College, in the class of 1867, with the degree of Civil Engineer. His studies at college were interrupted for nearly two years by service in the Union Army, in the War of the Rebellion, as a volunteer in the 16th Massachusetts Battery, which was a part of the Army of the Potomac. His army service was from March, 1864, until he was mustered out, in July, 1865.

After graduation, and until 1873, Mr. Earley was employed as Assistant Engineer on the construction of the Portland and Ogdensburg Railway, the Johnston and Glenerville Railway, and in other engineering work in New England.

In 1873 he went to the Cincinnati Southern Railway as Resident Engineer in charge of the first residency, Division "E," and remained there until the completion of the work, in 1876. His residency covered the work from Somerset, Kentucky, to the south side of the Cumberland River, some of the heaviest and most difficult work on that railway, which was noted for the excellency and permanency of its construction, especially of the masonry work. This work was a distinct departure from the ordinary railway practice in America, and elicited favorable comment from European technical journals of the period.

For a time after 1876, Mr. Earley was in the Government employ, on the improvement of the Tennessee River near Florence, Alabama.

From 1879 to the summer of 1881, he was engaged on the surveys of the Atchison, Topeka and Santa Fé Railway, in New Mexico and Arizona, being at times harassed by hostile Indians and most of the time working under the protection of a military escort furnished by the United States Government.

In the summer of 1881 Mr. Earley crossed the line into Mexico, and thereafter, and until the time of his death, played an important part in the development of that Republic, occupying many positions of importance and trust, among which were the following:

In 1881-82, he was Principal Assistant Engineer to the late W. R. Morley, M. Am. Soc. C. E., Chief Engineer of the Sonora Railway. From November, 1882, to April, 1883, he was Principal Assistant Engineer to Lewis Kingman, M. Am. Soc. C. E., then, as he is now, Chief Engineer of the Mexican Central Railway. From 1883 to 1887 Mr. Earley's work was near Guadalajara, Mexico, first as Chief Engineer of

* Memoir prepared by Francis Webster Blackford, M. Am. Soc. C. E.

the surveys and construction work in the rough country lying between there and Tepic, described by the late A. M. Wellington, M. Am. Soc. C. E., in his "Economic Theory of the Location of Railways"; and in surveying and building the Guadalajara Branch of the Mexican Central, of which he was Chief Engineer.

Subsequently, Mr. Earley was Chief Engineer of the Mexican Southern Railway, and located this road from Puebla to Oaxaca through a very rough and difficult country; the road, however, was built by an English construction company under its own engineer. He was also for a time Chief Engineer of the Cuernavaca Railway during its construction from the City of Mexico to the Balsas River, and later had charge of improvements and changes in the line of the Interoceanic Railway between Mexico City and Vera Cruz.

After 1902 Mr. Earley was engaged in a general engineering practice, with an office in the City of Mexico, and, at the time of his death, was Consulting Engineer and a Director of the Tlahualillo Cotton Company and also of the Guerrero Iron and Timber Company, which is developing large tracts of mineral and timber lands in the State of Guerrero, Republic of Mexico.

At Auburn, Maine, on January 15th, 1873, he married Miss Etta Sawyer, daughter of James Sawyer. This was followed by thirty-five years of congenial companionship, for Mrs. Earley accompanied her husband wherever it was practicable for a woman to go, and shared the hardships and wanderings incident to the lives of so many engineers. She survives him.

Mr. Earley was Chairman of the Entertainment Committee at the Annual Convention of the American Society of Civil Engineers in the City of Mexico in July, 1907, and soon after that meeting went to the United States, where he spent several months among his old friends and in the haunts of his boyhood in Massachusetts. This was a pleasant finale to a long and useful, and more or less arduous, life; for, on returning to Mexico, while the steamer was lying in the Port of Progreso, Yucatán, he suffered a stroke of paralysis from which he never recovered, and from the effects of which he died a few months later at the beautiful home in the City of Mexico which his industry had created and his good taste embellished with many evidences of refinement and culture.

Mr. Earley was a man of sterling character, and an engineer of large experience and good judgment, of which his work bears evidence. He had a cordial, sociable, generous disposition, and was an entertaining talker. His reminiscences of early experiences in the Western countries and in Mexico could always command attentive listeners.

Mr. Earley was elected a Member of the American Society of Civil Engineers on April 15th, 1876. He was also a member of the Masonic Fraternity and the Grand Army of the Republic.

OTHNIEL FOSTER NICHOLS, M. Am. Soc. C. E.*

DIED FEBRUARY 4TH, 1908.

The shock which is invariably felt at the tidings of the death of an active man in full health and vigor was experienced by the many friends of Othniel Foster Nichols when they learned that his life had gone out instantly and without warning in the early morning of February 4th, 1908, at his home, No. 42 Gates Avenue, Brooklyn, New York. All the day before he had been actively engaged in his professional duties at the Department of Bridges, had lunched with a group of his friends at the "Engineers' Table" at the Astor House, when he was as jovial and affable as was his wont.

He was descended from Rhode Island families which had taken a prominent and patriotic part in the affairs of the State, and whose ancestors settled in the Rhode Island Colony early in the seventeenth century. The son of Thomas Pitman and Lydia Foster Nichols, he was born at Newport, Rhode Island, on July 29th, 1845. Early in his life the family moved to Brooklyn, New York, where he attended the public schools, and in 1862 entered a machine shop, where he was employed for several years before beginning his professional training at the Rensselaer Polytechnic Institute, from which he was graduated with the degree of Civil Engineer in 1868. This practical and theoretical training, with a capacity for persistent and faithful work, made him a resourceful man who could readily adapt himself to any conditions. Immediately after graduation he was employed in the development of Prospect Park, Brooklyn.

Mr. Nichols took part in building the first elevated railroad in New York City, and taught mathematics in the evening course at Cooper Union, and in 1870 became a member of the engineering staff of Cooper, Hewitt and Company. In 1871 he went to Peru, where he was engaged for four years on the difficult work of locating and constructing the Chimbote and Huaraz and the Lima and Oroya Railways. Returning to the United States in 1876, he became Assistant Engineer and Superintendent of the Edgemoor Bridge Company in the contract of that Company for building the Sixth Avenue Elevated Railway in New York. Owing to delays in this work, caused by injunctions, he joined the engineering staff of the Department of Parks, which had charge of the work of developing the annexed district, now the Borough of the Bronx.

In 1878 he again went to South America, as Resident Engineer of the Madeira and Mamoré Railroad, in Brazil. This project ended in disaster, and Mr. Nichols was obliged to go to London in connection with litigation growing out of it. In 1879 he became Assistant Engi-

* Memoir prepared by Nelson P. Lewis, M. Am. Soc. C. E.

neer in the bridge shops of the New Jersey Steel and Iron Company, and in 1882 he became Resident Engineer on the Louisville and Nashville bridge across the Ohio River at Henderson, Kentucky. Upon the completion of this structure, in 1886, he became Chief Engineer of the water-works of Westerly, Rhode Island, but after a short time he resigned to take part again in the development of the Elevated Railway system of New York City, this time as Principal Assistant Engineer of the Suburban Rapid Transit Company in building what is now the Third Avenue line in the Borough of the Bronx. In 1888 he became Chief Engineer of the Brooklyn Elevated Railroad Company, in which capacity he designed and built a large part of the elevated lines belonging to that company, while in 1892 he became General Manager of the Company. When the "New East River Bridge Commission" was organized, in 1895, and Leffert L. Buck, M. Am. Soc. C. E., became Chief Engineer, Mr. Nichols was appointed Principal Assistant Engineer, and with this great structure, now known as the Williamsburg Bridge, he was closely identified from its beginning. In 1904 he became Chief Engineer of the Department of Bridges, which office he held for two years, and from 1906 until the abrupt end of his life he was Consulting Engineer of the same Department. He was therefore closely identified with the design and construction of all the great bridges across the East River, except the Brooklyn Bridge.

As this brief outline will show, Mr. Nichols' professional life was one of unusual activity, and, while his experience was varied, his time and energy were devoted chiefly to structural work, and during his later years to some of the world's greatest bridges.

Mr. Nichols showed keen interest and took an active part in professional organizations other than this Society. He became a Member of the Institution of Civil Engineers of Great Britain on February 2d, 1892, and in 1897 the Institution awarded him the Telford premium for his paper, "The Brooklyn Elevated Railway." He became a Member of the American Society of Mechanical Engineers in 1896. He was a Member of the Brooklyn Engineers' Club, and its President in 1904, a Member of The Municipal Engineers of the City of New York, and its first Vice-President. He was, at the time of his death and for some years previously, President of the Engineering Department of the Brooklyn Institute of Arts and Sciences. He was also a member of the Engineers' Club of New York, of the Crescent Athletic Club, and of the Municipal Club of Brooklyn, a trustee of the Polytechnic Institute of Brooklyn, and a member of the Vestry of St. Luke's Protestant Episcopal Church of Brooklyn. He was always active in the Alumni Association of the Rensselaer Polytechnic Institute, and in 1895 delivered the address to the graduating class of that institution.

Well read in the best literature, and an observant traveler, he was a

forceful writer and speaker, and as a companion he was entertaining and delightful, with a great fund of anecdote and reminiscence drawn from extensive travel and large acquaintance. Men of great professional attainments and of mental power are often lacking in those human qualities which make them loved as well as honored; but such qualities Mr. Nichols possessed in an unusual degree. A keen sense of honor in his business and professional relations; an unassumed pleasure in the success of his friends, and a warm sympathy in their reverses; a readiness to help those who needed assistance, not only by advice and good counsel drawn from his broad experience, but by giving of his substance; to his friends an unswerving loyalty which did not stop to consider his own personal interests—these are the qualities which made for him a host of friends, each one of whom felt a deep sense of personal loss when they heard of his untimely death.

It is not often that a man of mature years who has attained high professional standing shows a live interest in young men just beginning their life work. Many students and young engineers found in Mr. Nichols a ready, sympathetic, and patient listener to whom they could confide their perplexities and their disappointments, their hopes and ambitions, without danger of cynical comment or caustic criticism.

His active, useful, unselfish life, full of buoyant hope, and enriched by a rare capacity for the enjoyment of friendship and association with his fellows, came to its close without waning of power or impairment of vigor, and left a fine example of honor, loyalty and rectitude.

Mr. Nichols was elected a Member of the American Society of Civil Engineers on June 7th, 1876, was always active in the affairs of the Society, and served as a member of the Board of Direction in 1892 and 1893.

CHARLES FRANCIS POWELL, M. Am. Soc. C. E.*

DIED JULY 30TH, 1907.

The personal knowledge of the writer concerning General Powell covers only the four years that he was stationed at New London, while the writer was associated with him in the capacity of Assistant Engineer. Owing to General Powell's modesty, he rarely spoke of his previous career—almost never of his services during the Civil War—hence, in the preparation of this memoir, it has been necessary to depend upon official data and information furnished by others for the facts relating to his early life, and the writer wishes to make acknowledgment of the great aid and valuable information given him, especially concerning General Powell's early life and services during the Civil War, by his brother, Captain A. O. Powell, and by Major Harry Taylor, Corps of Engineers, in furnishing authentic records and dates concerning his service as an Officer of the Corps of Engineers. Without the aid of these gentlemen, the writer would have been unable to prepare this memoir.

Charles Francis Powell was born in Jacksonville, Illinois, on August 13th, 1843. His ancestors were Americans and his great-grandfather was a soldier in the Revolutionary War. Born without the advantages of wealth, Charles Francis Powell was, from boyhood, obliged to depend on his own exertions for support and advancement. While quite young he moved to Milwaukee, Wisconsin, where he obtained his early education in the public and private schools. When the Civil War broke out, and before he was eighteen years old, he began his military career, enlisting on May 10th, 1861, as a private in Company B, 5th Wisconsin Volunteer Infantry, and afterward being promoted to Corporal and then to Sergeant Major.

During the Civil War he saw much hard service and fierce fighting, and his war record is a brilliant one. His first service was in the suppression of the bank riot at Milwaukee in the summer of 1861; later in that year he was ordered East, and in September took part in the advance into Virginia. The following year he was with the Army of the Potomac in the Peninsula Campaign and took part in the Siege of Yorktown, the engagement at Lee's Mill, the Battle of Williamsburg, the engagements at Golding's Farm, Garnett's Farm, Savage's Station, and White Oak Swamp, and the Battle of Malvern Hill, all in less than three months. Later in that year he was on the field at the Second Battle of Bull Run and the Battle of Chantilly, but was not actively engaged. In September, 1862, he took part in the Antietam Campaign and on September 17th participated in the bloody Battle of Antietam. In November he was in the advance into Virginia near Harper's Ferry

*Memoir prepared by E. G. Verrill, M. Am. Soc. C. E.

and along the eastern base of the Blue Ridge. In 1863 he was on the Rappahannock opposite Fredericksburg; participated in the Gettysburg Campaign, and, after a forced march, took part in the great Battle of Gettysburg on July 2d, 3d, and 4th. On July 5th he was in an action with the enemy's rear guard; later in the month he was again in the advance into Virginia near Harper's Ferry, and in August took part in the enforcement of the second draft at New York and Albany. This was his last active service during the Civil War.

He was then only about twenty years old, but he was an experienced soldier, and had seen harder service and more of war and actual fighting than falls to the lot of many men who are soldiers all their lives and attain high rank. He had put in more than two years of the hardest sort of service, and had taken part in several of the greatest and most fiercely fought battles of the Civil War or of history, but he came out unscathed, though the writer has heard him say that he had two or more horses shot under him and his cap carried away by a rifle ball.

On September 29th, 1863, he was appointed a cadet at the United States Military Academy by President Lincoln. At this time the number of cadets at West Point was much reduced because of the disaffection of the southern Congressmen and the cadets appointed by them, and to fill these vacancies a number of cadets were appointed by President Lincoln from among deserving young soldiers of the Union Army, their appointments being from southern Congressional Districts which then had no cadets at West Point. Young Powell was among the soldiers thus appointed, his appointment being from a South Carolina District, and was stated to have been for "soldierly courage and ability, faithful and brave conduct and gallantry on the field of battle."

He passed his entrance examinations to West Point successfully, although, up to this time, his opportunities for education had been somewhat meager, and, for the two years and a half, or thereabouts, since he enlisted he must have had to give up study entirely. He was also under the disadvantage of entering after the academic year had begun, and hence was behind his class, but by hard work and perseverance he managed, not only to make up back work and keep up with his class, but to maintain a high stand. During his last year at West Point he met with an accident that nearly cost him his life. At cavalry drill his horse stumbled and fell with him and badly injured his knee; tetanus subsequently developed, and he was for a long time in the hospital, but finally recovered. Although he was left permanently lame, it was not sufficient to incapacitate him for duty. In spite of this accident, which would have made most men give up entirely, he graduated twelfth in a class of sixty-three and was commissioned, on June 17th, 1867, a Second Lieutenant in the Corps of Engineers. His sub-

sequent promotions in the Corps of Engineers were: First Lieutenant, April 23d, 1869; Captain, June 17th, 1881; Major, January 26th, 1895; Lieutenant Colonel, January 22d, 1904.

His first duty as an Engineer Officer was with the Engineer Battalion at Willets Point, New York, where he served from August, 1867, to May, 1871, as Company Officer, Post Quartermaster and Commissary, and Battalion Quartermaster, in the latter capacity for the last three years of his service at Willets Point. After leaving Willets Point he served as assistant on the geodetic survey of the Northern Lakes for nearly 8 years, and of the Mississippi River for nearly 2 years, to March, 1879, and as assistant to Major George L. Gillespie, Corps of Engineers, from April 11th, 1879, to October 26th, 1881, when he was appointed Engineer of the 13th Light-house District. This latter position he held until April 11th, 1888, at the same time being in charge of various defenses, river and harbor improvements, surveys, etc., in Oregon, Washington, and Idaho. The Cascades Canal and the commencement of the great jetty at the mouth of Columbia River were among the most important works of which he had charge at this time.

From May 31st, 1883, to November 18th, 1890, he was Secretary and Disbursing Officer of the Mississippi River Commission, and was charged with various duties connected with the improvement of that river; and from 1890 to April 27th, 1893, he was at Bismarck, North Dakota, and Sioux City, Iowa, in charge of the improvement of the Missouri River above Sioux City and the Yellowstone River, in Montana and North Dakota. At the close of the above tour of duty he was appointed Engineer Commissioner of the District of Columbia, Washington, D. C., which position he held until March 2d, 1897, when he was ordered to Pittsburg, Pennsylvania, and placed in charge of river and harbor work in that district, which included the improvement of Pittsburg Harbor, the Alleghany River, the Monongahela River slack-water navigation system, and various other engineering works. On January 8th, 1902, he assumed charge of the Connecticut District, comprising all river and harbor improvements in that State, the Pawcatuck River, between Rhode Island and Connecticut, and the defenses at the eastern entrance to Long Island Sound, his headquarters being at New London, Connecticut. Among the more important works in this district were the construction and equipment of various emplacements for modern high-power guns and mortars at the eastern entrance to Long Island Sound, the improvement of the Connecticut, Thames, and Housatonic Rivers, and the harbors of New Haven, New London, and Bridgeport, besides a number of other smaller rivers and harbors.

In the early part of 1906, while still stationed at New London, General Powell was taken with a severe illness, largely or wholly the result of over-work, but, with his extremely strict ideas with regard

to duty, he insisted upon still performing his official duties, even while confined to his room and bed. The writer will always remember one occasion in particular when he was directed to report in person to General Powell regarding certain official business, and found him at his home, barely able to sit up, and yet insistent upon talking over the matter in hand and making his decisions, although seized every few minutes with violent fits of coughing and scarcely able to hold a letter in his hand. Several times during his illness he recovered sufficiently so that, although contrary to the advice of his physician, he went to his office, but finally his condition became so serious that he was obliged to give up all official duties, and on March 8th, 1906, he was relieved of charge of the District. His condition had now become so critical that, upon the representations of his physician and family, his retirement was requested under the provisions of Section 1244, U. S. Revised Statutes. Before this was done, however, and on account of his splendid record during the Civil War, he was appointed Brigadier General, U. S. Army, dating from March 31st, 1906; his retirement took place three days later, April 3d, 1906.

After his retirement his health greatly improved for a time, although his family and friends had scarcely dared to hope for it, but the improvement was not permanent, and on July 30th, 1907, he died at St. Paul, Minnesota, where he had moved with his family after his retirement.

To quote from the General Orders of the Chief of Engineers announcing General Powell's death: "All duties entrusted to him were well and faithfully performed." Strict attention to duty and absolute adherence to the law and to his orders were cardinal principles of General Powell's life. Conscientious to the last degree, he always gave to his work the most unremitting personal attention, even to the smallest details, and it was this close personal application to the daily routine and minor details of his work, which most men in similar positions turn over to their assistants, that took much of his time and rendered his duties unusually arduous and confining.

Personally, General Powell was of an extremely kind, courteous, and pleasant disposition; unassuming and modest in manner and appearance, yet always an officer and a gentleman. To his assistants and those with whom he came into official contact he was always courteous, considerate, and approachable; to his family he was devoted.

He was married on May 17th, 1883, at Albany, Oregon, to Margaret Isabelle, daughter of James H. Foster, and is survived by his wife and six children.

General Powell was elected a Member of the American Society of Civil Engineers on October 3d, 1888.

GEORGE EDWARD THOMAS, M. Am. Soc. C. E.*

Died March 6th, 1908.

George Edward Thomas, born September 21st, 1841, in Haverfordwest, South Wales, Great Britain, was the son of John and Elizabeth (Walters) Thomas, and one of the youngest of ten children, seven girls and three boys. He received his early education in the Grammar School of Haverfordwest and the National School of Pembroke Dock. In July, 1856, he entered Her Majesty's Academy, Pembroke Dock Navy Yard, and, upon his graduation, served under Oliver Lang and John Inman Fincham, remaining until the spring of 1870.

He showed such marked energy and ability as a youth that, when resigning from Her Majesty's service, his commanding officer at first refused to grant his release.

On October 1st, 1864, Mr. Thomas married Miss Elizabeth Anne Jones, of Pembroke Dock; and, of their five children, two daughters and a son survive.

In July, 1870, Mr. Thomas moved to Chicago, Illinois, where he made his home until 1901, though his varied professional interests were largely in other fields.

From 1870 to 1877 he was with the Illinois Central Railroad under Mr. L. H. Clark, Chief Engineer. During this time he had charge of the construction of the superstructure for the first 3 000 ft. of breakwater for the Government Harbor, Chicago, and also the lakeshore protection for the Illinois Central Railroad. Later, he put in the cradles and fitted out the transfer boats for that railroad at Cairo, Illinois.

In 1877 Mr. Thomas entered the employ of William Sooy Smith, M. Am. Soc. C. E., a pioneer in the use of pneumatic caissons for foundations, and had charge of building and placing pneumatic caissons for a bridge over the Missouri River, at Glasgow, Missouri, for the Chicago and Alton Railroad.

In 1878 he was with Mr. C. E. Cooley on work for the Federal Government on the Missouri River at Nebraska City, Nebraska.

In 1879 he had charge of building and sinking pneumatic caissons for a bridge over the Missouri River at Plattsmouth, Nebraska, for the Burlington and Missouri River Railroad.

In 1880 he was called in on a bridge over the Red River in Arkansas for the St. Louis, Iron Mountain and Southern Railway.

In the fall of 1881 he was with General William Sooy Smith building a bridge for the Charleston and Savannah Railway at Hardyville, South Carolina. Later, he put in the foundations for a bridge over the Yazoo River, near Vicksburg, Mississippi.

* Memoir prepared by the Secretary from material furnished by Mr. Thomas' family.

Early in 1883 Mr. Thomas had charge of the installation of machinery for the Railroad Appliance Exposition in Chicago, of which the late Mr. John McGregor Adams, of the Adams and Westlake Company, of Chicago, was President. Later in the same year Mr. Thomas built the first electric railroad for traffic at the Louisville, Kentucky, Exposition, for Mr. Adams, this road having been an exhibit at the Railway Appliance Exposition in Chicago. Many professional men of the present day will remember the pioneer engine, the old "Judge." Comparing this first power-house of his construction with the great power-house on 59th Street, New York City, of which he was the Supervising Engineer, Mr. Thomas once wrote:

"The horse-power developed at Louisville, required to run the first electric railroad, equalled fifty; the horse-power developed at the 59th St. Power-House equals 142 000. The 50 horse-power at Louisville represented all the then used electrical power for street railroad purposes, and that at the 59th St. Power-House, not one-half the amount required in the City of New York; all this change and improvement in just twenty-one years."

In later years, apropos of Mr. Thomas' appointment as Supervising Engineer on this great New York City Subway power-house, Mr. Adams wrote:

"Am much pleased to know of the place that has sought you. It is a great compliment to your ability and skill, and I am satisfied no one could fill it more satisfactorily than you."

In 1883-84 Mr. Thomas had charge of building and sinking the pneumatic caissons used in the construction of the bridge over the Susquehanna River at Havre de Grace, Maryland, for the Baltimore and Ohio Railroad, and, for that railroad, he built and sunk the pneumatic caissons and had full charge of work on a bridge over the Schuylkill River at Philadelphia, Pennsylvania.

His many years of practical experience in subaqueous work made him expert in that branch of his profession. Concerning their mutual relations, General Sooy Smith writes:

"During the many years of his association with us, he occupied very important and difficult positions, and discharged his whole duty, in every one of them, with rare fidelity and ability, contributing largely to our success in the great works we carried out with his valuable assistance. He was indefatigable in his diligence, and his ingenuity and skill made him equal to every emergency."

In 1886 Mr. Thomas had charge of and sunk the shafts and drifted 300 ft. on the Canadian side, and 100 ft. on the American side of the St. Clair River at Port Huron, Michigan, for a preliminary tunnel under the river for the Grand Trunk Railroad.

He had charge of, built, and sunk the pneumatic caissons for the Huntington system, over the Ohio River at Cincinnati, Ohio, in 1887,

and also, later, made borings for the proposed bridge at Vallejo Junction, California, for the same system.

In August, 1888, Mr. Thomas assumed full charge of sinking a shaft for the Chapin Mining Company, at Iron Mountain, Michigan, for the Sooysmith-Poetsch Freezing Company. It was the first shaft sunk by the Poetsch method in the United States, and was brought to a successful issue in the early spring of 1889.

Charles Sooysmith, M. Am. Soc. C. E., writes, regarding their many years of intimacy:

"His courage was an inspiration that pushed many a great undertaking to speedy completion, and always helped those below and those with him to work with spirit and happiness."

In 1889-90 Mr. Thomas had charge of the construction of the Lubec Narrows Light House, at Lubec, Maine. This, together with the Shubenacadie Bridge at South Maitland, Nova Scotia, he considered as among his most difficult works, both being located on the Bay of Fundy.

Quoting Mr. Thomas, in a discussion of his paper "Piers of Midland Railway Bridge, Shubenacadie River, South Maitland, Nova Scotia,"*

"I made all the borings and all the surveys in New York harbor from Cortlandt street to Ft. Tompkins and up the Kill von Kull, for the proposed tunneling, etc., which is to be done under New York City."

This was for the Roberts-Corbin Syndicate, in 1892.

In 1892-93 Mr. Thomas had charge of, and sunk part of the caissons for the New Central Bridge over the Harlem River, New York City. He was transferred to take charge of sinking the pneumatic caissons under the Manhattan Life Building, New York City.

From 1894 to 1897 Mr. Thomas was engaged as a civil and consulting engineer in Chicago, Illinois, among his works being new piers for the Glasgow Bridge, construction contracts for the Lake Street "L" Railroad in Chicago, and the erection of a railroad bridge on the St. Joseph River, near Benton Harbor, Michigan.

From October, 1897, to October, 1898, he was called in to finish building ten piers and several abutments previously begun for the New York and Ottawa Railroad Bridge, at Cornwall, Ontario. At the conclusion of that contract he became associated with the Engineering Contract Company, and for them executed the following works: In the winter of 1898-99 the extensive repairs to the large dam of the Royal Electric Company at Chambly Canton, Quebec; and in the working seasons of 1899 and 1900 the construction of the piers for the Midland Railroad Bridge at South Maitland, Nova Scotia. This was a very difficult work, as it required the placing and sinking of five pneumatic caissons in a river having a rise and fall of tide of from

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* *Journal*, Western Society of Engineers, Vol. VI, 1901, p. 172.

24 to 32 ft., and a current of from 8 to 12 miles per hour. Operations had to be carried on between tides, owing to the fact that the incoming and outgoing water would carry out all unsecured work.

In the spring of 1900 he started the shafts for the Derby Lead Company, at Elvins, Missouri. In the summer of 1901 Mr. Thomas took charge of the construction of the foundations for the Marine Engineering Building at the Naval Academy, Annapolis, Maryland, for the Snare and Triest Company, of New York City. In the spring of 1902 he resigned, and accepted the position of engineer for Noel and Thomas, of Baltimore, Maryland, contractors for the construction of the Cadet Quarters at the Naval Academy.

In the fall of 1902 Mr. Thomas became Supervising Engineer for the Interborough Rapid Transit Company, of New York City, in the construction of its main power-house at 59th Street and North River, a work of immense magnitude, and costing about $7 000 000.

In his paper,* "Deep and Difficult Bridge and Building Foundations," Mr. Thomas describes many details of the difficult foundation work done by him, and some of his experience with the freezing process for sinking shafts through water-bearing soil is described in his discussion of the paper† "Freezing as an Aid to Excavation in Unstable Material," by James H. Brace, Assoc. M. Am. Soc. C. E.

On March 1st, 1906, he became supervising engineer for Mr. Francis H. Kimball, Architect, on the Brunswick Building, New York City, and in June he was transferred by Mr. Kimball to supervise the construction of the City Investing Building, a 34-story structure on Broadway and Cortlandt Street, New York City. He was engaged on this work at the time of his death. Concerning him, Mr. Kimball writes:

"His experience and skill in his profession were of great value, and I regret that his mantle cannot fall on someone who would be able to carry on his work. He has taught many what they know, but I know of no one so clear headed in time of need. Mr. Thomas leaves to his family a splendid reputation for skill in his calling, and a rugged, honest nature which stood out in bold relief among his fellows. No one knew him who did not respect him, and his loss is deplored by all."

Mr. Thomas always commanded the confidence of those he served and of those who served under him. The former trusted him because of his ability, because what he undertook he carried through successfully; while the latter, who worked with him, worked for him. He had their confidence because he was a leader who could not only direct but could practically carry out his plans. Men were always willing to follow him. In his hazardous undertakings he never ordered a man to go where he himself would not personally lead. He was a man of rare executive ability, and his advice and services were always in demand.

* *Journal*, Western Society of Engineers, Vol. I, 1896, p. 437.

† *Transactions*, Am. Soc. C. E., Vol. LII, 1904, p. 437.

Mr. Thomas was an active worker in church affairs, and at the time of his death was a steward and class leader in St. Andrew's Methodist Episcopal Church, New York City. He was also a Knight Templar. No man, however humble, who ever came to him for help or counsel went away empty-handed. He had a good word and a smile for all. His men not only respected but loved him.

In response to the many requests made to have him write the story of his life, his invariable reply was, "Oh, I'm too busy living; wait till I retire." He has written his life on the hearts of his fellow men, and on the towering tablets of his great and successful works.

He was elected a Member of the American Society of Civil Engineers on October 5th, 1898, and was for many years a member of the Western Society of Engineers.

TRANSACTIONS

OF THE

American Society of Civil Engineers

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